

CHAPTER 11

BANK PROTECTION

Chapter 11 Bank Protection
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Bank Protection

11.1 Introduction

11.1.1 Purpose

One of the hazards of placing a highway near a river or stream channel or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated and the proper type and amount of protection must be provided in the right locations.

This chapter provides procedures for the design of revetments to be used as channel bank protection and channel linings on larger streams and rivers. For small discharges, procedures presented in the Channel Chapter should be used. Emphasis in this chapter has been placed on wire-tied rock riprap and soil cement revetments due to their costs and past performance. Other channel stabilization methods such as spurs, guide banks, retard structures, longitudinal dikes and bulkheads are discussed in "Stream Stability at Highway Structures," Hydraulic Engineering Circular No. 20. Bank protection shall not be used to reduce the foundation depths of bridges.

11.1.2 Erosion Potential

Channel and bank stabilization is essential to the design of any structure affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for stabilization, is best accomplished through observation. A three level analysis procedure is provided in FHWA Hydraulic Engineering Circular No. 20. The three level analyses provides a rigorous procedure for determining the geomorphological characteristics, evaluating the existing conditions through field observations and determining the hydraulic and sediment transport properties of the stream.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information, or current site conditions. Aerial photographs, old maps and surveying notes and bridge design files and river survey data as well as gaging station records and interviews of long-time residents can provide documentation of any recent and potentially current channel movement or bank instabilities.

In addition, current site conditions can be used to evaluate stability. Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions which are indicative of instabilities may include tipping and falling of vegetation along the bank, cracks along the bank surface, the presence of slump blocks, fresh vegetation laying in the channel near the channel banks, deflection of channel flows in the direction of the bank due to some recently deposited obstruction or channel course change, fresh vertical face cuts along the bank, locally high velocities along the bank, new bar formation downstream from an eroding bank, local headcuts, pending or recent cutoffs, etc. It is also important to recognize that the presence of any one of these conditions does not in itself indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

11.1 Introduction (continued)

11.1.3 Symbols And Definitions

Table 11-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
AOS	Apparent opening size in filter cloth	mm
A	Coefficient used to determine the apparent opening size	
C	Coefficient, relates free vortex motion to velocity streamlines for unequal radius of curvature	-
C _u	Uniformity coefficient	
D ₅₀	The median bed material size	ft.
D ₁₅	The 15% finer particle size	ft.
D ₈₅	The 85% finer particle size	ft.
d _{avg}	Average flow depth in the main flow channel	ft.
d _s	Estimated probable maximum depth of scour	ft.
g	Gravitational acceleration (32.2 ft/s ²)	ft/sec ²
H	Wave height	ft.
k	Permeability	cm/sec or mm/sec
K ₁	Correction term reflecting bank angle	-
n	Manning's roughness coefficient	-
O ₉₅	Opening size in the geotextile material for which 95% of the openings are smaller	mm
Q _{mc}	Discharge in the zone of main channel flow	ft ³ /sec
R	Hydraulic radius	ft.
R _o	Mean radius of the channel centerline at the bend	ft.
S _f , S	Friction slope or energy grade line slope	ft/ft
SF	Stability factor	-
S _s , s	Specific gravity of the riprap material	-
T	Top width of the channel between its banks	ft.
V	Velocity	ft/sec
V _a	Mean channel velocity	ft/sec
W ₅₀	Weight of the median particle	lbs.
Z	Superelevation of the water surface	ft.
γ	Unit weight of the riprap material	lbs/ft ³
θ	Bank angle with the horizontal	degrees
Φ	Riprap material's angle of repose	degrees

11.2 Policy

Highway alignments and improvements often cross, encroach upon or otherwise require construction of a new channel or modification of the existing channel. It is necessary to protect the public, the highway investment and the environment from the natural reaction to the highway changes. ADOT policy requires that the facility, including bank protection, will perform without significant damage for flood and flow conditions experienced on the operational frequency recurrence interval. ADOT policy also requires that the facility, including bank protection, be evaluated regarding the hazard to adjacent people and property for flood and flow conditions experienced on a 100-year recurrence interval. The facility, to the maximum extent possible, shall perpetuate natural drainage conditions thus protecting and maintaining the environment.

11.3 Design Criteria

To provide an acceptable standard level of service, ADOT traditionally employs the pre-established operational design frequencies that are based on the importance of the transportation facility to the system and the allowable risk for that facility. This is true for revetment protection. However, although the operational design flow frequency standards represent the consensus on reasonable values, actual design must consider consequences of any other event that may produce a more severe hydraulic condition. Under certain conditions, it may be appropriate to establish the level of risk allowable for a site and to design to that level. In addition, design standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be addressed in the design.

Minimum Criteria:

Design Discharge -- Operational Frequency Flow

Freeboard at Operational Design Flow--1 foot plus velocity head

However, the designer should be aware that in some instances, a lower discharge may produce hydraulically worse conditions with respect to riprap stability. It is suggested that several discharge levels be evaluated to ensure that the design is adequate for all discharge conditions up to that selected as the design discharge for structures associated with bank protection.

The selection of the particular bank protection method used will depend on several factors. Among the factors to be considered are:

- constructibility
- risk of failure
- consequence of failure
- maintenance
- environmental impacts
- cost

11.4 Bank And Lining Failure Modes

11.4.1 Potential Failures

Prior to designing the bank stabilization plan, the designer must be aware of common erosion mechanisms and revetment failure modes, and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Bank failure modes include:

- particle erosion,
- translational slide,
- modified slump, and
- slump.

11.4.2 Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

A frequent particle erosion failure is the loss of the underlying material resulting in undermining and eventual collapse of the revetment protection. Usually the underlying material is lost through the revetment or piped under the toe of the revetment protection. This failure is very common in and extremely damaging to rigid types of protective linings. Providing a suitable filter, either natural or fabrics in conjunction with hydrostatic relief features will prevent this failure.

Another frequent type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence that in turn increases the tractive stresses placed on the protective layer, underlying layers, and the natural bank material beyond the revetment. This failure area needs to receive special attention since extension of the protective feature usually only moves, not eliminates, the failure.

11.4.3 Translation Slide

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the bank that extend parallel to the channel. As the slide progresses, the lower part of bank separates from the upper part, and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

11.4 Bank And Lining Failure Modes (continued)

11.4.4 Modified Slump

The failure of riprap referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

11.4.5 Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

11.5 Revetment Types

11.5.1 Common Types

The types of revetment used for bank protection and stabilization by ADOT include:

- wire-tied rock (gabions),
- soil cement,
- paved lining (concrete slope pavement),
- grouted rock,
- rock riprap,
- grouted fabric.

A type of revetment that has been used in the past, but is not currently recommended are sand/cement bags. Descriptions of each of these revetment types are included below.

11.5.2 Wire-Tied Rock

Wire-tied, or gabion, revetment consist of wire mesh elements filled with rock. Wire-tied rock revetments are generally of two types distinguished by shape: wire mattresses, or gabions. In wire-tied mattress designs, the individual wire mesh units are laid end-to-end and side-to-side to form a mattress layer on the channel bed or bank. The baskets comprising the mattress are subdivided and generally have a depth dimension that is much smaller than their width or length. Gabions, on the other hand, are typically rectangular or trapezoidal in shape. They are more equal-dimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. Gabions may be stacked vertically to increase the protected height.

11.5 Revetment Types (continued)

11.5.3 Soil Cement

Soil-cement generally consists of a dry mix of sand and cement and admixtures batched either in a central mixing plant or on-site. It is usually transported, placed by equipment capable of producing the width and thickness required and compacted to the required density. Control of the moisture and time after introduction of the mixing water is critical. Curing is required. This results in a rigid protection. Soil-cement can be placed either as a lining or in stepped horizontal layers. The stepped horizontal layers are extremely stable provided toe scour protection or embedment has been incorporated in the design.

11.5.4 Concrete Slope Pavement

Concrete pavement revetments (concrete slope pavement) are cast-in-place on a prepared slope to provide the necessary bank protection. Concrete pavement is a rigid revetment which does not conform to changes in bank geometry due to a removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. The loss of even small sections of the supporting embankment can lead to complete failure of the revetment system.

11.5.5 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. Grouted rock is a rigid revetment; it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted rock is not extremely strong; therefore, the loss of even a small area of bank support can cause failure of large portions of the revetment.

11.5.6 Riprap

Rock riprap is described as a layer or facing of loose rock, dumped or hand-placed. Materials other than rock are also referred to as riprap; for example, rubble, broken concrete slabs and preformed concrete shapes (slabs, blocks, rectangular prisms, etc.). These materials are similar to rock in that they can be hand-placed or dumped onto an embankment to form a flexible revetment.

11.5.7 Grouted Fabric Slope Pavement

Grouted fabric slope pavement revetments are constructed by injecting sand-cement mortar between two layers of double-woven fabric that has been positioned on the slope to be protected. Mortar is injected into this fabric envelope either underwater or in-the-dry. The fabric enclosure prevents dilution of the mortar during placement underwater. The two layers of fabric act first as the top and bottom form to hold the mortar in place while it hardens. This fabric, to which the mortar remains tightly bonded, then acts as tensile reinforcing to hold the mortar in place on the slope. These revetments are analogous to slope paving with reinforced concrete. The bottom layer of fabric acts as a filter cloth underlayment to prevent

11.5 Revetment Types (continued)

11.5.7 Grouted Fabric Slope Pavement (continued)

loss of soil particles through any cracks which may develop in the revetment as a result of soil subsidence. Often greater relief of hydrostatic uplift is provided by weep holes or filter points that are normally woven into the fabric and remain unobstructed by mortar during the filling operation.

11.5.8 Sand-Cement Bags

A method that has been used in the past, but is not recommended is sand-cement bag revetment. Sand-cement bag revetment generally consist of a dry mix of sand and cement placed in a burlap or other suitable bag. They are hand-placed in contact with adjacent bags. They require firm support from the protected bank. Usually a filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility, low tensile strength and is susceptible to damage particularly on flatter slopes where the area of contact between the bags is less.

11.6 Design Concepts

11.6.1 Introduction

Design concepts related to the design of bank protection are discussed in this section. Subjects covered in this section include flow types, section geometry, flow in channel bends, flow resistance and extent of protection.

11.6.2 Flow Types

Open channel flow can be classified from three points of reference. These are:

- uniform, gradually varying, or rapidly varying flow;
- steady or unsteady flow; and
- subcritical or supercritical flow.

Design relationships presented in this chapter are based on the assumption of uniform, steady, subcritical flow. These relationships are also valid for gradually varying flow conditions. While the individual hydraulic relationships presented are not in themselves applicable to rapidly varying, unsteady, or supercritical flow conditions, procedures are presented for extending their use to these flow conditions (see the Channel chapter for more details related to channel design). Rapidly varying, unsteady flow conditions are common in areas of flow expansion, flow contraction and reverse flow. These conditions are common at and immediately downstream of bridges. Supercritical or near supercritical flow conditions are common at bridge constrictions and on steep sloped channels.

11.6 Design Concepts (continued)

11.6.2 Flow Types (continued)

Non-uniform, unsteady and near supercritical flow conditions create stresses on the channel boundary which are significantly different from those induced by uniform, steady, subcritical flow. These stresses are difficult to assess quantitatively. The stability factor method of riprap design presented in Section 11.7.1 provides a means of adjusting the final riprap design (which is based on relationships derived for steady, uniform, subcritical flow) for the uncertainties associated with these other flow conditions. The adjustment is made through the assignment of a stability factor. The magnitude of the stability factor is based on the level of uncertainty inherent in the design flow conditions.

11.6.3 Section Geometry

Design procedures presented in this chapter require as input channel cross-section geometry. The cross section geometry is necessary to establish the hydraulic design parameters (such as flow depth, top width, velocity, hydraulic radius, etc.) required by the bank protection design procedures, as well as to establish a construction cross section for placement of the revetment material. When the entire channel perimeter is to be stabilized, the selection of the appropriate channel geometry is only a function of the desired channel conveyance properties and any limiting geometric constraints. However, when the channel bank alone is to be protected, the design must consider the existing channel bottom geometry.

The development of an appropriate channel section for analysis is very subjective. The intent is to develop a section that reasonably simulates a **worst-case condition** with respect to bank protection stability. Information that can be used to evaluate channel geometry includes current channel surveys, past channel surveys (if available) and current and past aerial photos. In addition, the effect channel stabilization will have on the local channel section must be considered.

The first challenge arises in the selection of determining the channel profile to be used: Often it is intended to use the existing channel bottom profile. A single channel profile is usually not enough to establish the design cross section. In addition to current channel surveys, historic surveys can provide valuable information. A comparison of current and past channel surveys at the location provides information on the general stability of the site, as well as a history of past channel geometry changes. Often, past surveys at a particular site will not be available. If this is the case, past surveys at other sites in the vicinity of the design location may be used to evaluate past changes in channel geometry.

11.6.4 Flow In Channel Bends

Flow conditions in channel bends are complicated by the distortion of flow patterns in the vicinity of the bend. In long, relatively straight channels, the flow conditions are uniform and symmetrical about the centerline of the channel. However, in channel bends, the centrifugal forces and secondary currents produced lead to non-uniform and non-symmetrical flow conditions. Special consideration must be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends.

The following equations for estimating bend scour in sand-bed channels is presented in the “Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona” (1989):

11.6 Design Concepts (continued)

11.6.4 Flow In Channel Bends (continued)

$$Z_{bs} = \frac{0.0687 Y_{\max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} [2.1 \{ \sin^2(a/2) / \cos(a) \}^{0.2} - 1] \quad (11-1)$$

Where:

Z_{bs} = Bend scour component of total scour depth, in feet;

= 0 when $r_c/T \geq 10.0$, or $a < 17.8^\circ$ deg.

= computed value when $0.5 < r_c/T < 10.0$, or $17.8^\circ < a < 60^\circ$ deg

= computed value at $a = 60^\circ$ when $r_c/T \leq 0.5$ or $a > 60^\circ$

Y_{\max} = Maximum depth of flow immediately upstream of bend, in feet.

Y_h = Hydraulic depth of flow immediately upstream of bend, in feet.

V_m = Average velocity of flow immediately upstream of bend, in Ft/sec.

S_e = Energy slope immediately upstream of bend, in ft/ft.

a = angle formed by projection of channel centerline from point of curvature to a point which meets tangent line to the outer bank of the channel, in degrees (see below)

For a simple circular curve, the following relationship exists between the bend angle, a , the ratio of centerline radius, r_c and the channel top width, T .

$$r_c/T = \frac{\cos a}{4 \sin^2(a/2)}$$

Where:

r_c = Radius of curvature along centerline of channel, in feet.

T = Channel top width, in feet.

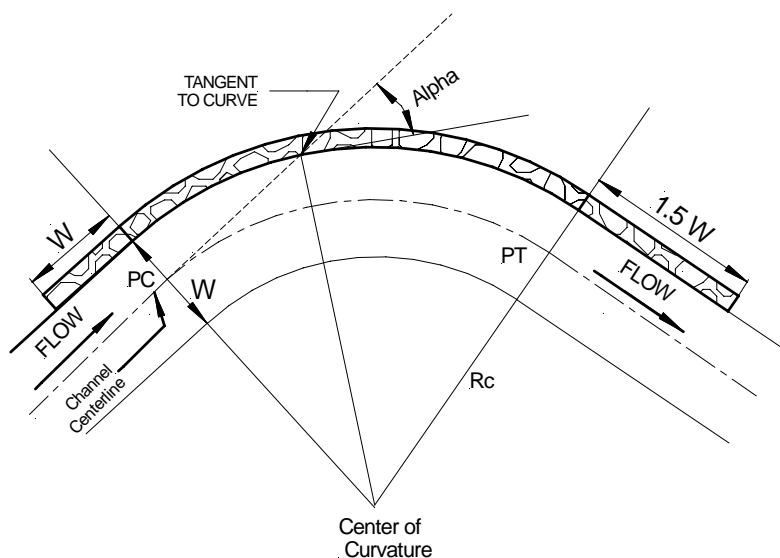


Figure 11-1 Flow at a bend

11.6 Design Concepts (continued)

11.6.4 Flow In Channel Bends (continued)

If the bend deviates significantly from a simple circular curve, the curve should be subdivided into a series of circular curves. The bend scour should be calculated for each segment of the curve based on the angle applicable to that segment. As the interaction of bend scour with other occurring channel forming activities is not known, the bend scour shall be considered to act independent of the other activities and shall be added to any other considered changes in the channel shape. Bend scour shall be considered to have a lateral extent downstream of the channel PT a distance x estimated as:

$$x = \frac{0.6Y}{n}^{1.17} \quad (11-2)$$

where:

x = Distance from downstream end of channel curvature(point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;

n = Manning's roughness coefficient;

Y = depth of flow, in feet; (use the maximum depth of flow, exclusive of scour)

Superelevation of flow in channel bends is another important consideration in the design of revetments. Although the magnitude of superelevation is generally small when compared with the overall flow depth in the bend (usually less than 1 foot) it should be considered when establishing free board limits for bank protection schemes on sharp bends. The magnitude of superelevation at a channel bend may be estimated for subcritical flow by the following equation:

$$Z = C [(V_a^2 T)/(gR_o)] \quad (11.3)$$

where: Z = superelevation of the water surface, ft

C = coefficient that relates free vortex motion to velocity streamlines for unequal radius of curvature,

V_a = mean channel velocity, ft/sec

T = water-surface width at section, ft

g = gravitational acceleration – 32.2 ft/sec²,

R_o = the mean radius of the channel centerline at the bend, ft

The coefficient C has been evaluated by the U.S. Geological Survey (USGS) and ranges between 0.5 and 3.0, with an average value of 1.5.

11.6.5 Flow Resistance

The hydraulic analysis performed as a part of the design process requires the estimation of Manning's roughness coefficient. Physical characteristics upon which the resistance equations are based include the channel base material, surface irregularities, variations in section geometry, bed form, obstructions, vegetation, channel meandering, flow depth and channel slope. In addition, seasonal changes in these factors must also be considered. See Channels Chapter, for a discussion of the selection of Manning's n values.

11.6 Design Concepts (continued)

11.6.6 Extent Of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

11.6.6.1 Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection plan is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criterion, it demands serious consideration. Review of existing bank protection sites has revealed that a common misconception in streambank protection is to provide protection too far upstream and not far enough downstream.

One criterion for establishing the longitudinal limits of protection required is illustrated in Figure 11-1. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criterion is based on analysis of flow conditions in symmetric channel bends under ideal laboratory conditions.

Real-world conditions are rarely as simplistic. In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (such as bridge abutments) might already be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. In addition, the magnitude or nature of the flow event might only cause erosion problems in a very localized portion of the bend, requiring that only a short channel length be stabilized. Therefore, the above criteria should only be used as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Since the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

11.6 Design Concepts (continued)

11.6.6 Extent Of Protection (continued)

11.6.6.2 Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

Design Height

The design height of a riprap installation should be equal to the design highwater elevation plus some allowance for freeboard. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. Some such factors include:

- superelevation in channel bends,
- hydraulic jumps, and
- flow irregularities due to piers, transitions and flow junctions.

In addition, erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash and debris in the channel, and aquatic or other growth in the channels should be considered when setting freeboard heights.

As indicated, there are many factors that must be considered in the selection of an appropriate freeboard height. As a minimum, it is recommended that a freeboard elevation of 1 foot plus velocity head be used. The Federal Emergency Management Agency requires 3 ft. for levee protection and 4 ft. at bridges for the 100-year flood. When computational procedures indicate that additional freeboard may be required, the greater height should be used. In addition, it is recommended that the designer consult existing records, and interview persons who have knowledge of past conditions when establishing the necessary vertical extent of protection required for a particular revetment installation.

Toe Depth

The undermining of revetment toe protection has been identified as one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation as well as natural scour and fill processes. In application, the depth of scour, d_s , should be measured from the lowest elevation in the cross section. It should be assumed that the low point in the cross section may eventually move adjacent to the protection (even if this is not the case in the current survey). A minimum embedment depth of 5 feet should be used in natural channels.

11.7 Design Guidelines

11.7.1 Rock Riprap

This section contains design guidelines for the design of rock riprap. Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap.

11.7.1.1 Bank Slope

A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, normally the maximum recommended face slope is 2H:1V. To be stable under an identical flow conditions, a revetment with a steep slope will need larger sizes and greater thickness than one with a flatter slope.

11.7.1.2 Rock Size

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. In the following sections, relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

Particle Erosion

Two methods or approaches have been used historically to evaluate a material's resistance to particle erosion. These methods are the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

Design Relationship

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in Equation 11.6. The design relationship is based on the assumption of uniform, gradually varying flow. Figure 11-7 presents a graphical solution to Equation 11-6.

$$D_{50} = 0.001C V_a^3 / (d_{avg}^{0.5} K_1^{1.5}) \quad (11.6)$$

Where: D_{50} = the median riprap particle size, inches
 V_a = the average velocity in the main channel, ft/sec
 C = Correction factor (described below)
 d_{avg} = the average flow depth in the main flow channel, ft
 K_1 = Factor for bank slope, see below

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

K_1 is defined as:

$$K_1 = [1 - (\sin^2 \Theta / \sin^2 \Phi)]^{0.5} \quad (11.7)$$

Where: Θ = the bank angle with the horizontal

Φ = the riprap material's angle of repose

Equation 11.7 can be solved using Figures 11-8 and 11-9. The average flow depth and velocity used in Equation 11.6 are main channel values. The main channel is defined as the area between the channel banks (see Figure 11-2 below).

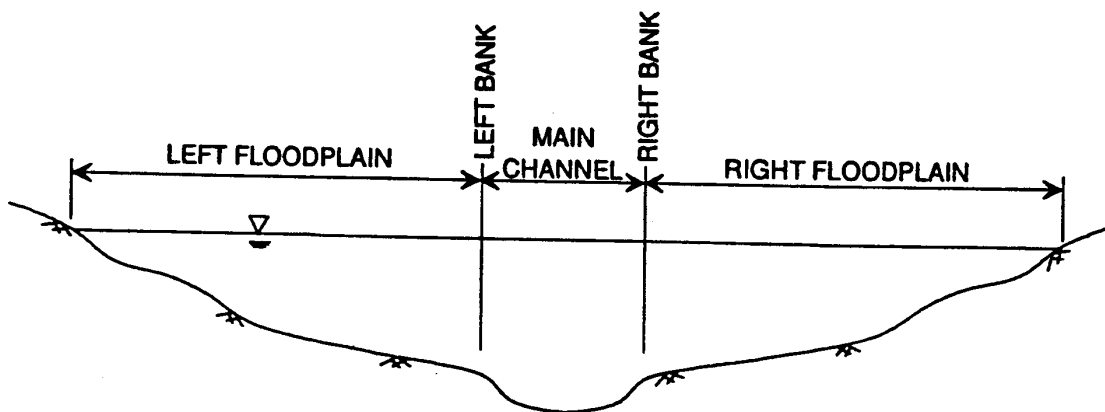


Figure 11-2 Definition Sketch; Channel Flow Distribution

Equation 11.6 is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equations 11.8 and 11.9 present correction factors for other specific gravities and stability factors.

$$C_{sg} = 2.12 / (S_s - 1)^{1.5} \quad (11.8)$$

Where: S_s = the specific gravity of the rock riprap

$$C_{sf} = (SF / 1.2)^{1.5} \quad (11.9)$$

Where: SF = the stability factor to be applied.

The correction factors computed using Equations 11.8 and 11.9 are multiplied together to form a single correction factor C . This correction factor, C , is then multiplied by the riprap size computed from Equation 11.6 to arrive at a stable riprap size. Figure 11-10 provides a solution to Equations 11.8 and 11.9 using correction factor C .

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

The stability factor, SF, used in Equations 11.6 and 11.9 requires additional explanation. The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of Equation 11.6.

The Stability Factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. The basic relationship in Equation 11.6 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris impacts, or the cumulative effect of high shear stresses and forces from wind generated waves. The Stability Factor is used to increase the design rock size when these conditions must be considered. Table 11-3 presents guidelines for the selection of an appropriate value for the stability factor.

Table 11-3 Guidelines For The Selection Of Stability Factors

<u>Condition</u>	<u>Stability Factor Range</u>
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.....	1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width > 10); Impact from waves or floating debris moderate.....	1.21 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris; Significant wind generated waves (1-2 ft.); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.....	1.61 - 2.0

Riprap shall have a Stability Factor (SF) equal to or greater than 1.25 for culver outlets and ditches; 1.40 for roadway embankments and channels; and 1.6 for bank protection at bridges.

Application

Application of the relationship in Equation 11.6 is limited to uniform or gradually varying flow conditions that are in **straight or mildly curving channel reaches of relatively uniform cross section**. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels with sharp bends and steep slopes, and in the vicinity of bridge piers and abutments. To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels, and at bridge abutments, it is recommended that Equation 11.6 be used with appropriate adjustments in velocity and/or stability factor as outlined below.

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

Bend Factor

For channels with sharp bends, $r/w \leq 7$, the following correction factor shall be applied to the Stability Factor.

$$D = 2.65/(r/w)^{0.5} \text{ with } D > 1.0$$

Where: r = radius of channel at center line, ft
 w = channel top width, ft

NOTE: $D = 1$ for $r/w > 7$.

Steep Slopes

Flow conditions in steep sloped channels are rarely uniform, and are characterized by high flow velocities and significant flow turbulence. In applying Equation 11.6 to steep slope channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep sloped channels, it is recommended that Equation 11.10 be used to determine the channel roughness coefficient. It is also important to thoughtfully consider the guidelines for selection of stability factors as presented in Table 11-3.

On high gradient streams it is extremely difficult to obtain a good estimate of the median bed material size. For high gradient streams with slopes greater than 0.002 and bed material larger than 0.2 ft. (gravel, cobble, or boulder size material), it is recommended that the relationship given in the following equation be used to evaluate the base Manning's n .

$$n = 0.39 S_f^{0.38} R^{-0.16} \quad (11.10)$$

Where: S_f = friction slope, ft/ft
 R = hydraulic radius, ft

11.7.1.3 Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The gradation limits should not be so restrictive that production costs would be excessive. Table 11-4 presents suggested guidelines for establishing gradation limits.

11.7 Design Guidelines (continued)

Rock Riprap (continued)

Table 11-4 Rock Riprap Gradation Limits

Percent of Gradation Smaller Than	Stone Size Range, Ft.	Stone Weight Range (lb)
100	1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}
85	3.0 W_{50} to 5.0 W_{50}	2.0 W_{50} to 2.75 W_{50}
50	1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}
15	0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}

It is recognized that the use of a four point gradation as specified in Table 11-4 might in some cases be too harsh a specification for some sources. If this is the case, the 85% specification can be dropped. In most instances, a uniform gradation between D_{50} and D_{100} will result in an appropriate D_{85} .

11.7.1.4 Layer Thickness

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness.

- It should not be less than the spherical diameter of the D_{100} (W_{100}) stone, or less than 2.0 times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.
- It should not be less than 12 inches for practical placement.
- The thickness determined by either 1 or 2 should be increased by 50% when the riprap is placed underwater to provide for uncertainties associated with this type of placement.
- An increase in thickness of 6 to 12 inches, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice, or by waves from boat wakes, wind, or bedforms.

11.7 Design Guidelines (continued)

Rock Riprap (continued)

Table 11-5 Riprap Gradation Classes

Riprap Class	Rock Size ¹ (ft)	Rock Size ² (pounds)	Percent of Riprap Smaller Than
Facing	1.30	200	100
	1.00	75	50
	0.40	5	10
Light	1.80	500	100
	1.30	200	50
	0.40	5	10
0.25 ton	2.25	1000	100
	1.80	500	50
	1.00	75	10
0.50 ton	2.85	2000	100
	2.25	1000	50
	1.80	500	5
1 ton	3.60	4000	100
	2.85	2000	50
	2.25	1000	5
2 ton	4.50	8000	100
	3.60	4000	50
	2.85	2000	5

1 Assuming a specific gravity of 2.65.

2 Based on AASHTO gradations.

11.7 Design Guidelines (continued)

11.7.1.5 Filter Design

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement and permits relief of hydrostatic pressures within the soils. Geotextile filters have replaced granular filters in most applications.

For rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15% particle size, (D_{15}) of the coarser layer to the 85% particle (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15% of the coarser material to the 15% particle size of the finer material should exceed 5 but be less than 40. These requirements are stated as

$$\frac{D_{15, \text{(coarser layer)}}}{D_{85, \text{(finer layer)}}} < 5 < \frac{D_{15, \text{(coarser layer)}}}{D_{15, \text{(finer layer)}}} < 40$$

The first test is intended to prevent piping, the right portion of the second test provides for adequate permeability and the right portion provides for uniformity criteria. In addition to the particle size ratios, the grain size curves should approximately parallel to minimize the infiltration of the fine material for the finer layer to the coarser layer. Figure 11-14 can be used to plot the gradation curves.

Geotextile Filters

Considerations for use of geotextile filters include the following:

- Geotextiles have widely variable hydraulic properties and must be designed based on project specific conditions and performance requirements.
- Geotextile filter performance is sensitive to construction procedures.
- Special installation and inspection procedures may be necessary when using geotextile filters.

Geotextile Filter Design

The design of geotextile filters should consider the following performance areas.

- Soil retention (piping resistance)
- Permeability
- Clogging
- Survivability

It is extremely desirable that individual site conditions and performance requirements be established in conjunction with the geotextile design. Generalized geotextile requirements should be used only on very small or non-critical/non-severe installations where a detailed analysis is not warranted. The American Association of State Highway and Transportation Officials (AASHTO) has developed materials and construction specifications (AASHTO Specification M-288) for routine, non-critical/non-severe geotextile applications. Details of geotextile filter design, for all levels of project severity and criticality are presented in the Federal Highway Administration publication, "Geosynthetic Design and Construction Guidelines,"

11.7 Design Guidelines (continued)

11.7.1.5 Filter Design (continued)

(FHWA-HI-95-038). This reference provides detailed guidance on specifying and installing geotextiles for a variety of transportation applications. The American Society for Testing Material Committee D-35 has developed standard testing procedures for approximately 35 general, index and performance properties of geosynthetics. These standard test procedures are recommended for use in design and specifications when using geosynthetics. See Appendix A for Geotextile Design information.

The following design steps are necessary for geotextile design in riprap and other permanent erosion control applications:

Step 1 - Evaluate the application site (determine if the application is critical or severe).

Step 2 - Obtain and test soil samples (perform grain size analysis and permeability tests).

Step 3 - Evaluate possible armor material and placement procedures.

Step 4 - Calculate anticipated reverse flow through the erosion control system.

Step 5 - Determine geotextile requirements:

- a. Soil Retention
- b. Permeability/Permittivity
- c. Clogging
- d. Survivability

Step 6 - Estimate cost and prepare specifications.

Geotextile Installation Procedures

To provide good performance, a properly selected cloth should be installed with due regard for the following precautions:

- Grade area and remove debris to provide a smooth, fairly even surface.
- Place geotextile loosely, laid with the machine (generally roll) direction in the direction of anticipated water flow or movement.
- Seam or overlap the geotextile as required.

11.7 Design Guidelines (continued)

11.7.1.5 Rock Riprap (continued)

- The maximum allowable slope on which a riprap-geotextile system can be placed is equal to the lowest soil-geotextile friction angle for the natural ground or stone-geotextile friction angle for cover (armor) materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability. For slopes greater than 2.5H:1V, special construction procedures will be required.
- For streambank and wave action applications, the geotextile must be keyed in at the bottom of the slope. If the system cannot be extended a few feet above anticipated high water level, the geotextile should also be keyed in at the crest of the slope.
- Place the revetment (cushion layer and/or riprap) over the geotextile width, while avoiding puncturing it.

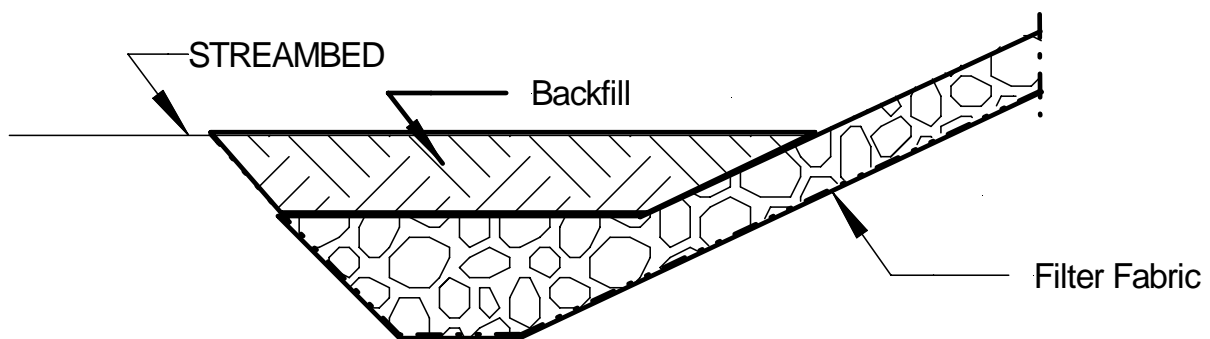


Figure 11-3 Geotextile Filters

11.7.1.6 Edge Treatment

The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 11-4. The upstream flank and the downstream flank are illustrated in section A-A of this figure. Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed to be below the anticipated scour as illustrated in Figure 11-5.

11.7 Design Guidelines (continued)

11.7.1.6 Edge Treatment (continued)

The toe material should be placed in a toe trench along the entire length of the riprap blanket. Where the toe material cannot be placed at the desired depth, the riprap blanket should terminate in a thick, stone toe at the lowest level possible (see alternate design in Figure 11-6). **It shall be located below the channel thalweg.**

The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will) the stone in the toe will launch into the eroded area as illustrated in Figure 11-8. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 2H:1V.

The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2H:1V) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternate location can be used when the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 11.6.7

11.7.1.7 Construction Considerations

The construction considerations related to the construction of riprap revetments include bank slope or angle, bank preparation and riprap placement.

Bank Preparation

The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 in. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions. In addition, any large boulders or debris found buried near the edges of the revetment should be removed.

Riprap Placement

The common methods of riprap placement are machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Re-handling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone, and can result in a rough revetment surface. Stone should not be dropped from an excessive height as this may result in the same undesirable conditions.

11.7 Design Guidelines (continued)

11.7.1.7 Construction Considerations (continued)

Riprap placement by dumping with spreading is satisfactory provided the required layer thickness is achieved. Riprap placement by dumping and spreading is the least desirable method as a large amount of segregation and breakage can occur.

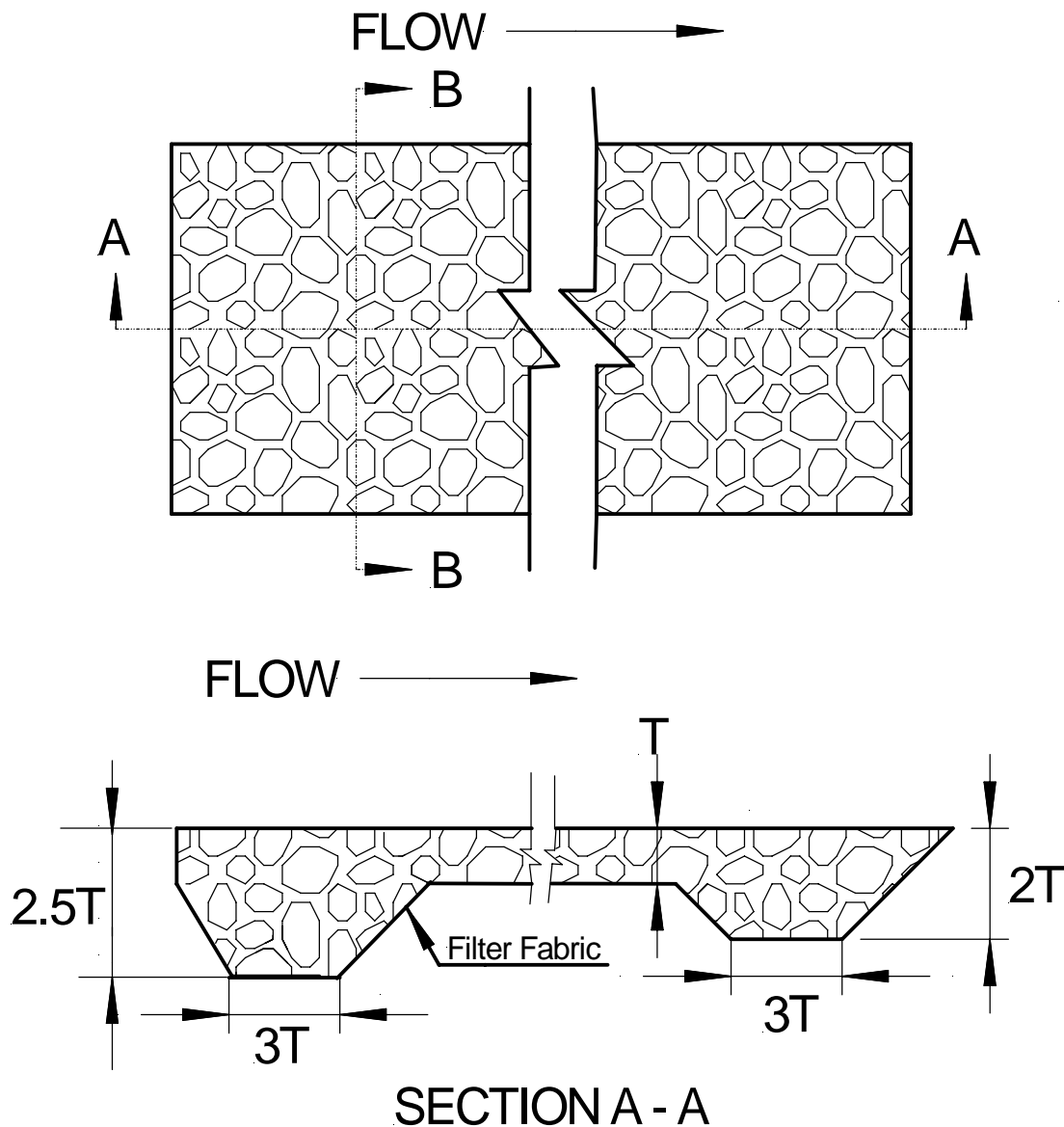


Figure 11-4 Typical Riprap Installation: Plan and Flank Details

11.7 Design Guidelines (continued)

11.7.1.7 Construction Considerations (continued)

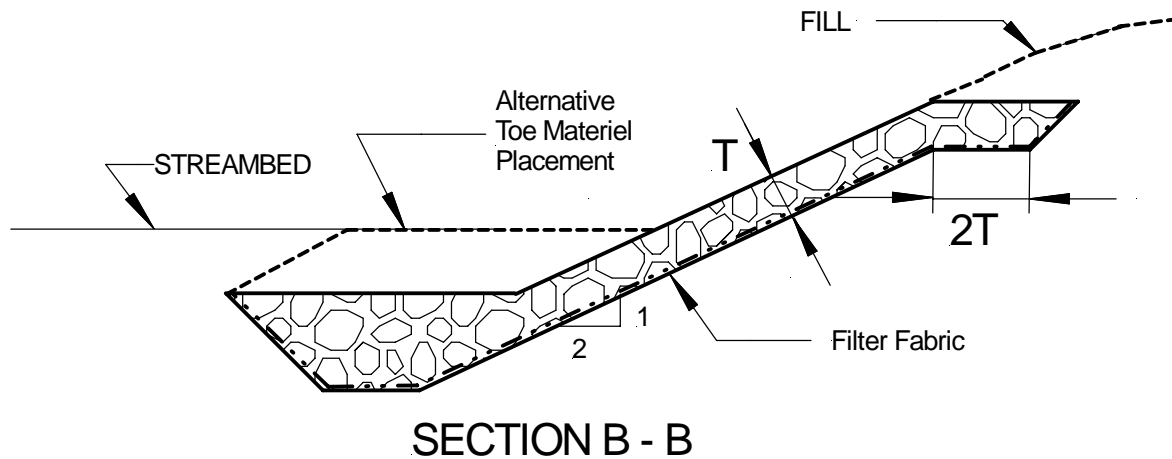


Figure 11-5 Typical Riprap Installation: End View (Bank Protection Only)

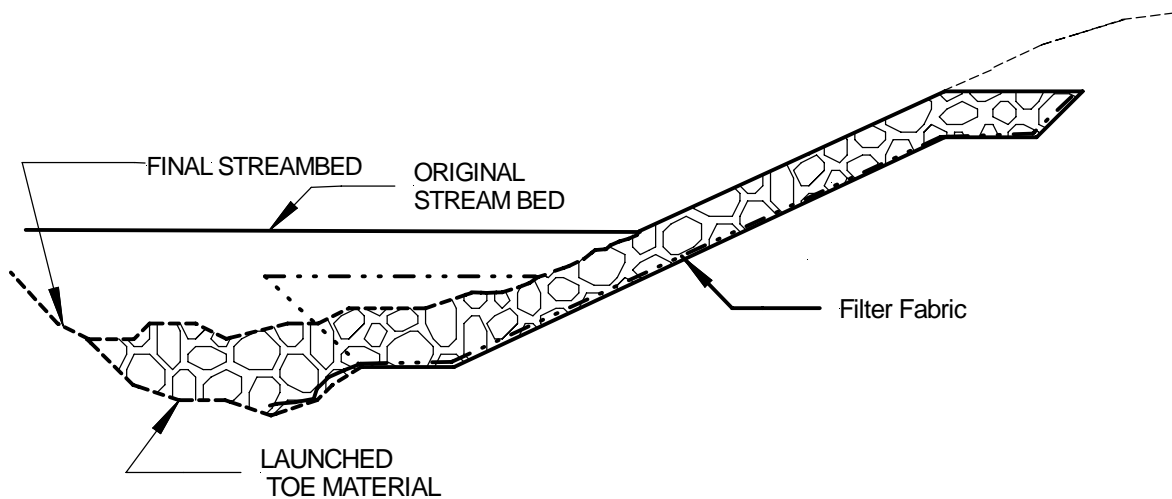


Figure 11-6 Launching Of Riprap Toe Material

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

11.7.1.8 Design Procedure

Rock riprap design procedure outlined in the following sections is comprised of three primary sections: preliminary data analysis, rock sizing and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections. Figure 11-13 provides a useful format for recording data at each step of the analysis.

Preliminary Data

Step 1 Compile all necessary field data including (channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.).

Step 2 Determine design discharge.

Step 3 Develop design cross section(s). Note: The rock sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.

Step 4 Compute design water surface.

- (a) When evaluating the design water surface, Manning's n should be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient n (Section 11.6.6).
- (b) If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, use design procedures from the Open Channels Chapter.
- (c) If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.
- (d) Any backwater analysis conducted must be based on conveyance weighing of flows in the main channel, right bank and left bank.

Step 5 Determine design average velocity and depth.

- (a) Average velocity and depth should be determined for the design section in conjunction with the computations of step 4. In general, the average depth and velocity in the main flow channel should be used.
- (b) If riprap is being designed to protect channel banks, abutments, or piers located in the floodplain, average floodplain depths and velocities should be used.

11.7 Design Guidelines (continued)

11.7.1.8 Design Procedure (continued)

Step 6 Compute the bank angle correction factor K_1 (Equation 11.7, Figures 11-8 and 11-9).

Step 7 Determine riprap size required to resist particle erosion (Equation 11.6, Figure 11-7).

- (a) Initially assume no corrections.
- (b) Evaluate correction factor for rock riprap specific gravity and stability factor ($C = C_{sg} C_{sf}$).
- (c) If designing riprap for piers or abutments see Bridge Chapter.

Step 8 Review Design. If entire channel perimeter is being stabilized, and an assumed D_{50} was used in determination of Manning's n for backwater computations, return to step 4 and repeat steps 4 through 7.

Step 9 Select final D_{50} riprap size, set material gradation (see section 11.7.1.3 and Figure 11-10), and determine riprap layer thickness (see section 11.7.1.4).

Step 10 Define limits of protection.

- (a) Determine longitudinal extent of protection required (section 11.6.7).
- (b) Determine appropriate vertical extent of revetment (section 11.6.7).

Step 11 Design filter layer (section 11.7.1.5, Figure 11-12).

- (a) Determine appropriate filter material size, and gradation.
- (b) Determine layer thickness.

Step 12 Design edge details (flanks and toe) (section 11.7.1.6).

11.7 Design Guidelines (continued)

11.7.1.7 Rock Riprap (continued)

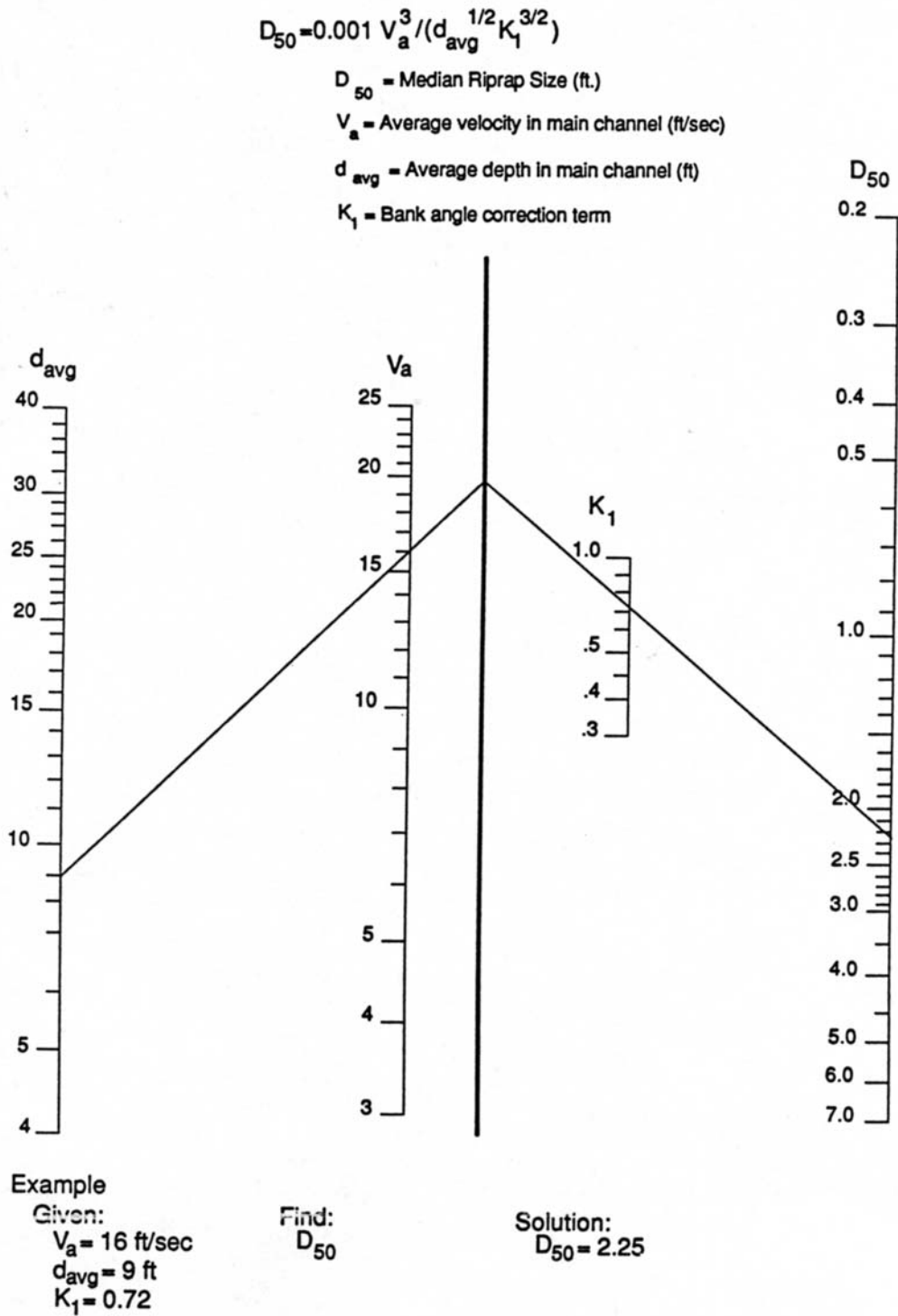
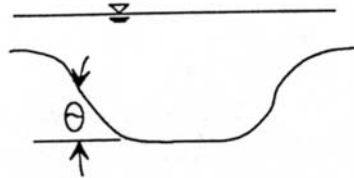


Figure 11-7 Riprap Size Relationship

11.7 Design Guidelines (continued)

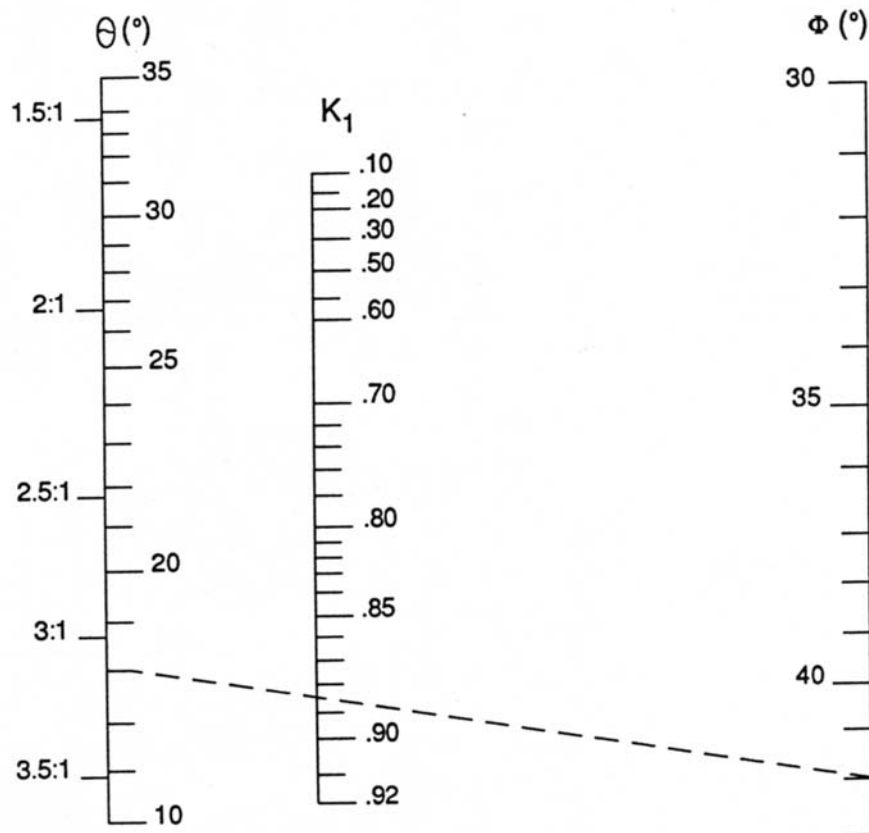
11.7.1 Rock Riprap (continued)



$$K_1 = \left[1 - \frac{\sin^2 \Theta}{\sin^2 \Phi} \right]^{0.5}$$

Θ = Bank angle with horizontal

Φ = Material angle of repose
(See chart 4)



Example

Given:
 $\Theta = 16^\circ$
Very Angular
 $D_{50} = 1.5$ ft.

Find:
 K_1

Solution:
 $\Phi = 42^\circ$
 $K_1 = 0.885$

Figure 11-8 Bank Angle Correction Factor (K_1) Nomograph

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

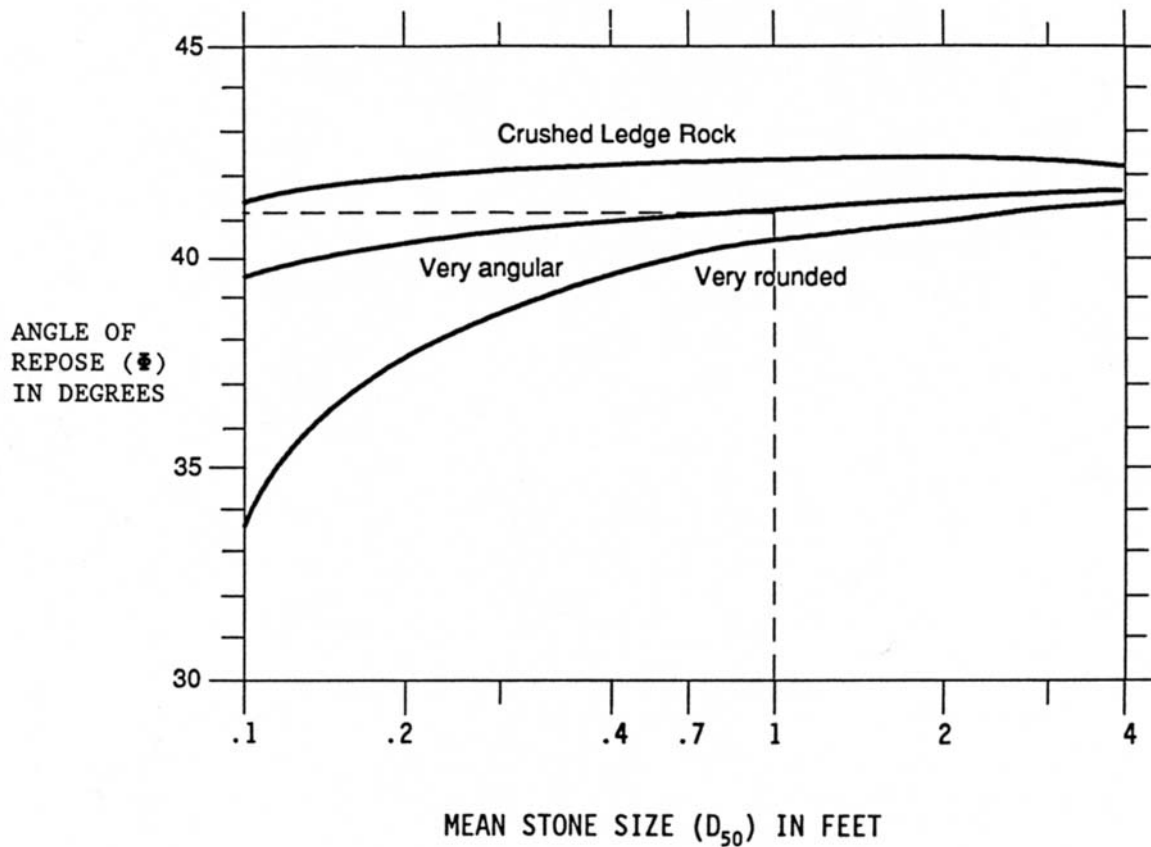


Figure 11-9 Angle Of Repose Of Riprap In Terms Of Mean Size And Shape Of Stone

11.7 Design Guidelines (continued)

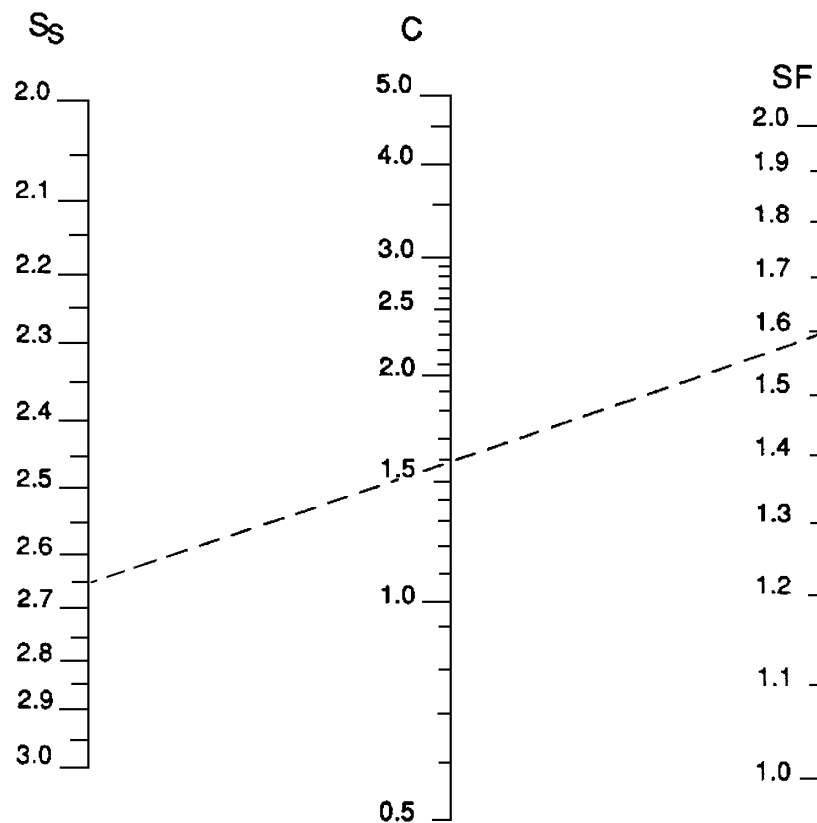
11.7.1 Rock Riprap (continued)

$$C = 1.61 SF^{1.5} / (S_s - 1)^{1.5}$$

CORR = D_{50} CORRECTION FACTOR

SF = STABILITY FACTOR

S_s = SPECIFIC GRAVITY OF ROCK



Example:

Given:
 $S_s = 2.65$
 SF = 1.60

Find:
 C

Solution:
 C = 1.59

Figure 11-10 Correction Factor For Riprap Size

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

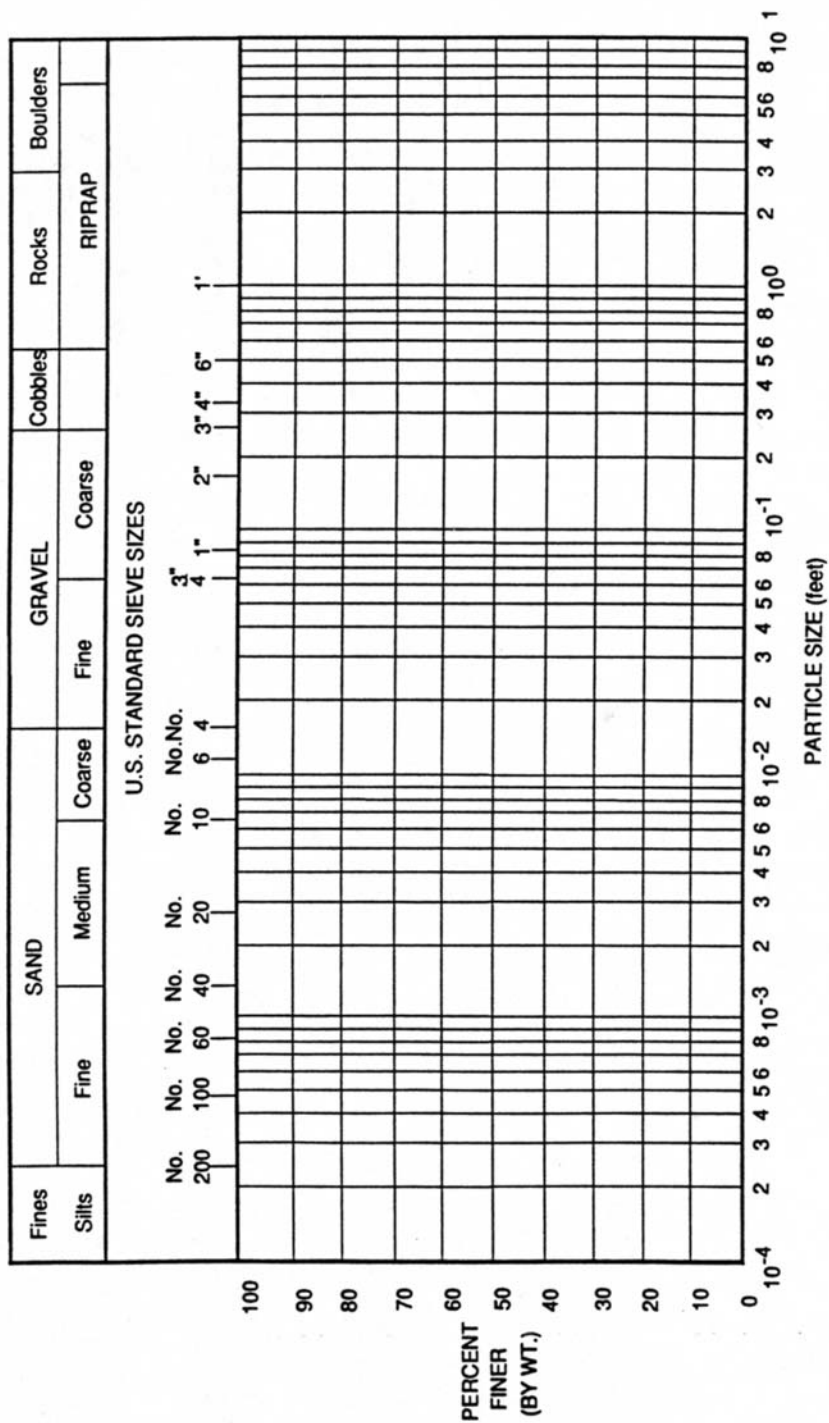


Figure 11-11 Material Gradation Chart

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

FILTER DESIGN			
Project Name: _____		Project No.: _____	
Subject _____		Page ____ of ____	
By _____	Date _____	Checked By _____	Date _____

GRANULAR FILTER

LAYER	DESCRIPTION	D ₁₅	D ₈₅	RATIO:			
				$\frac{D_{15} \text{ RIPRAP}}{D_{85} \text{ SOIL}}$	<5	$\frac{D_{15} \text{ RIPRAP}}{D_{15} \text{ SOIL}}$	<.40
	1						

DATA SUMMARY			
LAYER DESCRIPTION	D ₁₅	D ₈₅	THICKNESS

FABRIC FILTER

PHYSICAL PROPERTIES CLASS _____

HYDRAULIC PROPERTIES:

PIPING RESISTANCE , 50% PASSING #200 AOS<0.6mm

PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

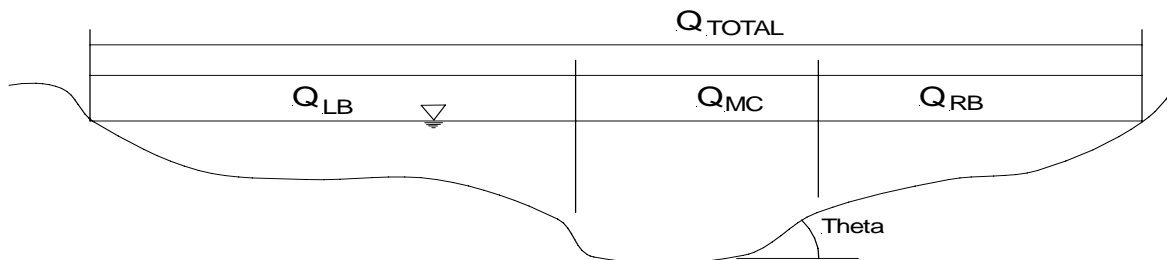
SELECTED FABRIC FILTER SPECIFICATIONS: _____

Figure 11-12 Filter Design

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

Rock Riprap			
Project Name: _____		Project No.: _____	
Subject _____		Page ____ of ____	
By _____	Date _____	Checked By _____	Date _____



Discharge DATA			
Q_{total}	Q_{LB}	Q_{MC}	Q_{RB}

DATA SUMMARY	Bank	Bed
Area, Ft^2		
Velocity, (ft./sec.)		
Depth, d_a , ft		
Theta, θ		
Phi, ϕ		
K_1		
D_{50} , ft		
SF		
S_g , g		
C		
$C_{P/A}$		
D_{50}		

RIPRAP Characteristics		
SIZE		
D50		
Class		
THICKNESS		
2D50		
D100		
USE		
Gradation	Percent Finer	Size
	100	
	50	
	5-10	

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY >	

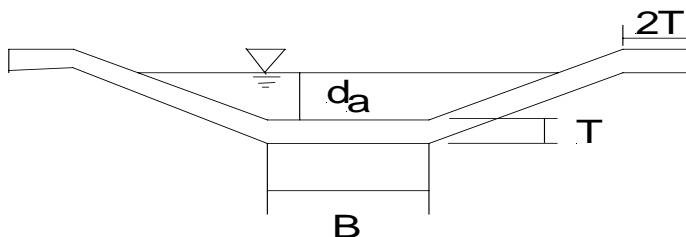


Figure 11-13 Riprap Size Form (Riprap Size Particle Erosion)

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

11.7.1.9 Design Examples

The following design examples illustrate the use of the design methods and procedures outlined above. Two examples are given; Example 1 illustrates the design of a riprap lined channel section. Example 2 illustrates the design of riprap as bank protection. In the examples, the steps correlate with the design procedure outline presented above. Computations are also shown on appropriate figures.

Example 1

A 1250 ft channel reach is to be realigned to make room for the widening of an existing highway. Realignment of the channel reach will necessitate straightening the channel and reducing its length from 1250 ft to 1000 ft. The channel is to be sized to carry 5,000 cfs within its banks. Additional site conditions are as follows:

- flow conditions can be assumed to be uniform or gradually varying;
- the existing channel profile dictates that the straightened reach be designed at a uniform slope of 0.0049;
- the natural soils are gap graded from medium sands to coarse gravels giving the following distribution.
 $D_{85} = 0.105 \text{ ft}$, $D_{50} = 0.064 \text{ ft.}$, $D_{15} = 0.0045 \text{ ft.}$
 $K \text{ (permeability)} = 3.5 \times 10^{-2} \text{ cm/s}$
- Available rock riprap has a specific gravity of 2.65 and $D_{50} = 12 \text{ inches}$.

Design a stable trapezoidal riprap lined channel for this site. Design figures used to summarize data in this example are reproduced in Figures 11-14 and 11-17.

Step 1 Compile Field Data

- See given information for this example.
- Other field data would typically include site history, geometric constraints, roadway crossing profiles, site topography, etc.

Step 2 Design Discharge

- Given as 5,000 ft³/sec.
- Discharge in main channel equals the design discharge since entire design discharge is to be contained in channel as specified.

Step 3 Design Cross Section

- As specified, a trapezoidal section is to be designed.
- Initially assume a trapezoidal section with 20 ft bottom width and 2H: 1V side slopes (see Figure 11-14).

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 4 Compute Design Water Surface

- (a) Determine roughness coefficient ($n = 0.04$)
- (b) Compute flow depth

Assume $R = 7$ feet

- Solve Manning's equation for normal depth.
 $Q = (1/n) A R^{2/3} S^{1/2}$
 $d = 11.8$ ft.
- Compute hydraulic radius to compare with the assumed value used in Step 4(a) (use computer programs, available charts and tables, or manually compute).
 $R = A/P$
 $R = 514.5/72.8$
 $R = 7.1$ ft. which is approximately equal to R (assumed) therefore,
 $d = 11.8$ ft OK

Step 5 Determine Design Parameters

$$A = 11.8(11.8(4) + 20 + 20)/2 = 514.5 \text{ ft}^2$$

$$V_a = Q/A = 5000/514.5 = 9.7 \text{ ft/sec}$$

$$d_a = d = 11.8 \text{ (uniform channel bottom)}$$

$$K_1 = 1 \text{ for bed}$$

Step 6 Bank Angle Correction Factor

$$\Theta = 2H: 1V$$

$$\Phi = 41^\circ \text{ (from Figure 11-11)}$$

$$K_1 = 0.73 \text{ (from Figure 11-10)}$$

Step 7 Determine riprap size (see section 11.7.1.8)

- (a) Using Figure 11-15
 for channel bed $D_{50} = 0.28$ ft
 for channel bank $D_{50} = 0.43$ ft.
- (b) Riprap specific gravity = 2.65 (given)
 (uniform flow, little or no uncertainty in design)
 Stability factor = 1.2 (Table 11-3)
 $C = 1$ (Figure 11-10)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

(c) no piers or abutments to evaluate for this example, therefore

$$C_{p/a} = 1$$

(d) Corrected riprap size:

For channel bed

$$D'_{50} = D_{50} = 0.28 \text{ ft.}$$

For channel banks

$$D'_{50} = D_{50} = 0.43 \text{ ft.}$$

Step 8 Not applicable

Step 9 Select Design Riprap Size, Gradation and Layer Thickness

D_{50} size: Recommend AASHTO Face Class riprap, table 11-5

$D_{50} = 0.95 \text{ ft.}$ (for entire perimeter)

Layer thickness (T):

$$T = 2 D_{50} = 2(0.95) = 1.9 \text{ ft.}, \text{ or } T = D_{100} = 1.3 \text{ ft.}$$

Use $T = 2.0 \text{ ft}$

Step 10 Define limits of protection.

Longitudinal Extent of Protection

Riprap lining to extend along entire length of straightened reach plus some additional upstream and downstream distance.

Vertical Extent of Protection

Riprap entire channel perimeter to top-of-bank.

Step 11 Filter Layer Design

(a) Filter material size:

$$<5 \frac{D_{15} \text{ [coarser layer]}}{D_{85} \text{ [finer layer]}} \quad \text{and} \quad \frac{D_{15} \text{ [coarser layer]}}{D_{15} \text{ [finer layer]}} < 40$$

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

For the riprap to soil interface:

$$\frac{D_{15} [\text{riprap}]}{D_{85} [\text{soil}]} = \frac{0.6}{0.105} = 6 > 5$$

and

$$\frac{D_{15} [\text{riprap}]}{D_{15} [\text{soil}]} = \frac{0.6}{0.0045} = 133 > 40$$

Therefore, a filter fabric is needed.

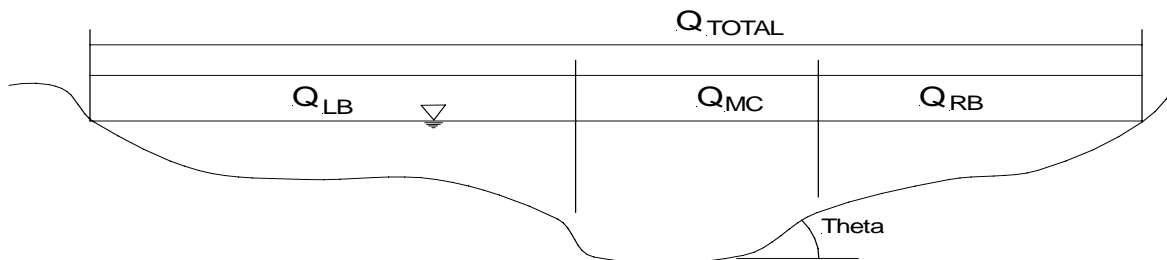
Step 12 Edge Details

Line entire perimeter; edge details as per Figure 11-6 (also see sketch on Figure 11-18).

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Rock Riprap			
Project Name: <u>Design Example</u>		Project No.: <u>ADT064</u>	
Subject <u>Channel Lining</u>		Page <u>1</u> of <u> </u>	
By <u> </u>	Date <u> </u>	Checked By <u> </u>	Date <u> </u>



Discharge DATA			
Q _{total}	Q _{LB}	Q _{MC}	Q _{RB}
5,000 cfs			

DATA SUMMARY	Bank	Bed
Area, Ft ²	--	514.5
Velocity, (ft./sec.)	--	9.7
Depth, d _a , ft	--	11.8
Theta, θ	26.5 (2:1)	--
Phi, ϕ	41	41
K ₁	.73	1.0
D ₅₀ , ft	0.43	0.28
SF	1.2	1.2
S _g , g	2.65	2.65
C	1	1
C _{P/A}	n/a	n/a
D ₅₀	0.43	0.28

RIPRAP Characteristics		
SIZE		
D ₅₀	0.95	
Class	Facing	
THICKNESS		
2D ₅₀	1.90	
D ₁₀₀	1.30	
USE:	2.0	
Gradation	Percent Finer	Size
	100	1.30
	50	0.95
	5-10	0.40

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY >	

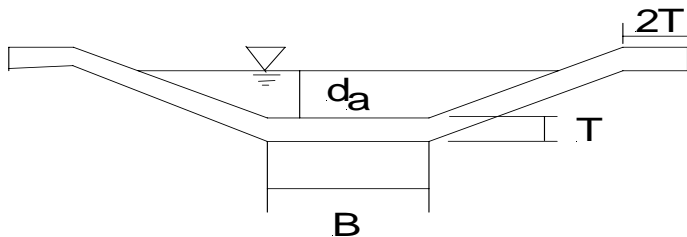


Figure 11-14 Riprap Size Form (Example 1)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

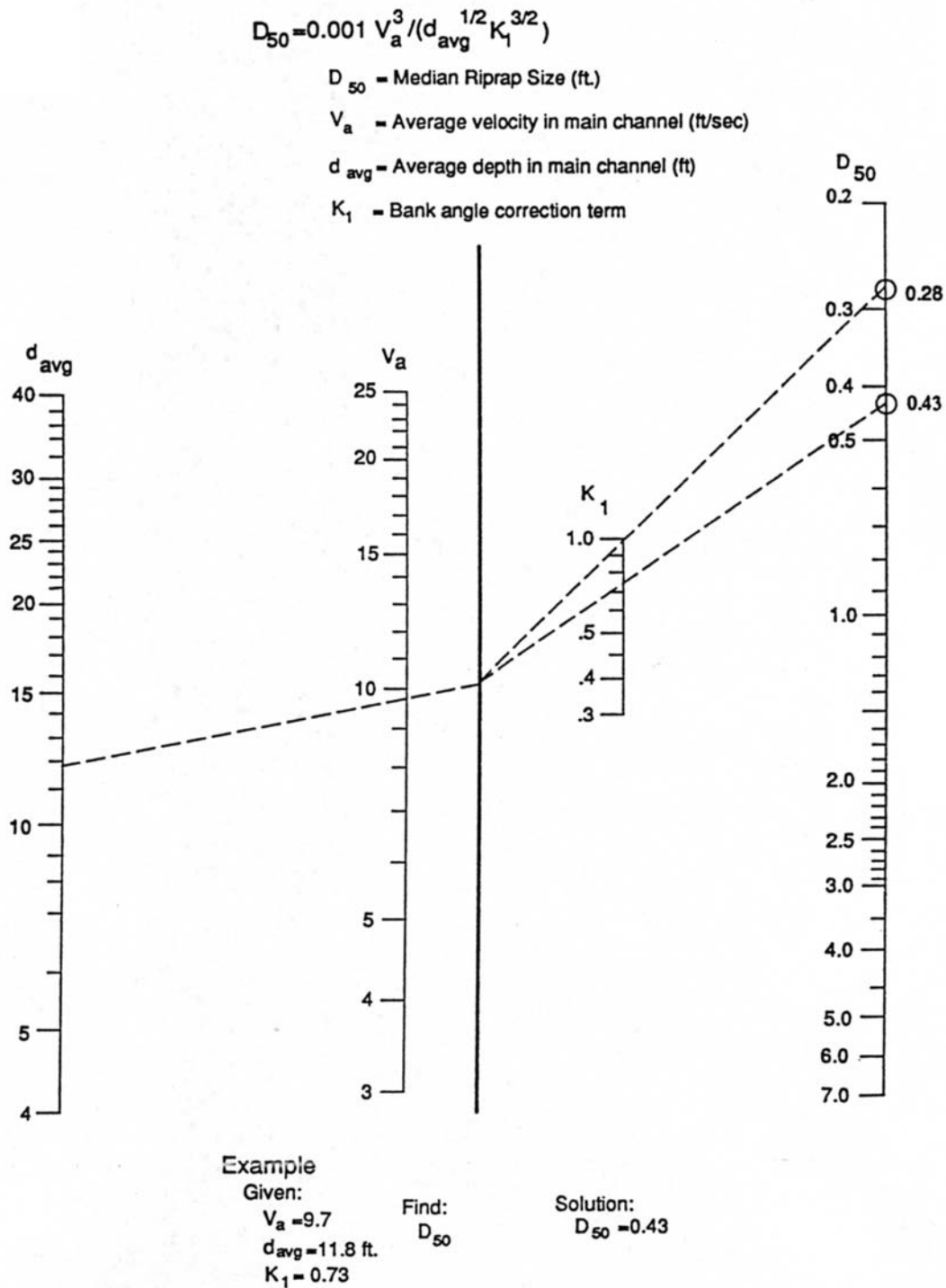


Figure 11-15 Riprap Size Relationship (Example 1, Step 7)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

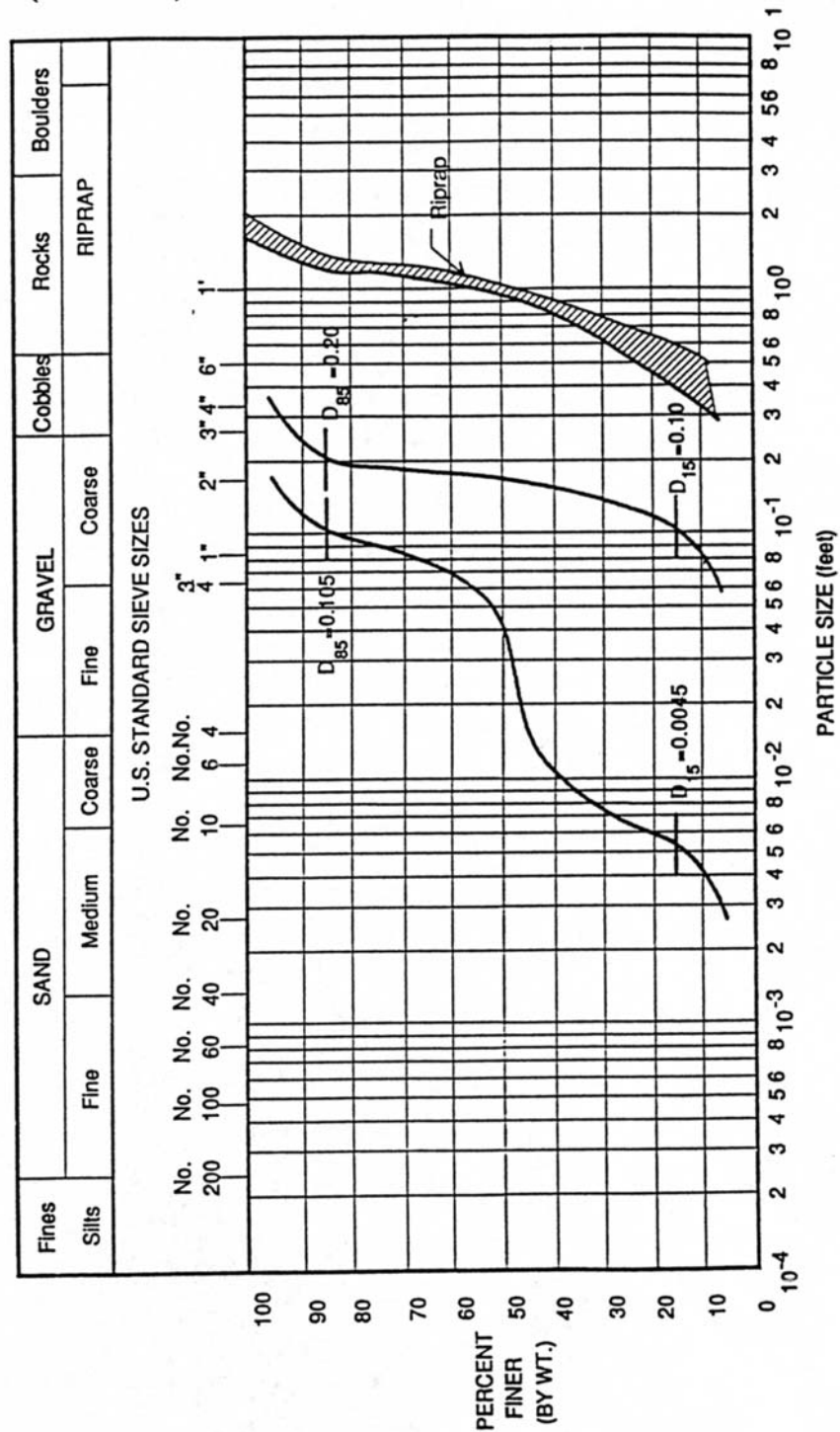


Figure 11-16 Material Gradation

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

FILTER DESIGN			
Project Name: _____		Project No.: _____	
Subject _____		Page ____ of ____	
By _____	Date _____	Checked By _____	Date _____

GRANULAR FILTER

LAYER	DESCRIPTION	D ₁₅	D ₈₅	RATIO:			
				$\frac{D_{15} \text{ RIPRAP}}{D_{85} \text{ SOIL}}$	<5	$\frac{D_{15} \text{ RIPRAP}}{D_{15} \text{ SOIL}}$	<40
	Riprap	0.60					
	Soil	0.0045	0.105				
				6	No	133	No

DATA SUMMARY			
LAYER DESCRIPTION	D ₁₅	D ₈₅	THICKNESS
Riprap	0.60		
Soil	0.0045	0.105	

FABRIC FILTER

PHYSICAL PROPERTIES CLASS _____

HYDRAULIC PROPERTIES:

PIPING RESISTANCE , 50% PASSING #200 AOS<0.6mm

PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: _____

Figure 11-17 Filter Design

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Example 2

The site illustrated in Figure 11-18 and discussed below is migrating laterally towards Route 1. Design a riprap revetment to stabilize the active bank erosion at this site.

The process of developing an appropriate channel geometry is illustrated in Figure 11-18. Figure 11-18 illustrates the location of the design site at position "2" along Route 1. The section illustrated in Figure 11-18c was surveyed at this location, and represents the current condition. No previous channel surveys were available at this site. However, data from several old surveys were available in the vicinity of a railroad crossing upstream (location 1). Figure 11-18b illustrates these survey data. The surveys indicate that there is a trend for the thalweg of the channel to migrate within the right half of the channel. Since location 1 and 2 are along bends of similar radii, it can be reasonably assumed that a similar phenomenon occurs at location 2. A thalweg located immediately adjacent to the channel bank reasonably represents the worst case hydraulically for the section at location 2. Therefore, the surveyed section at location 2 is modified to reflect this. In addition, the maximum section depth (located in the thalweg) is increased to reflect the effect of stabilizing the bank. The maximum depth in the thalweg is set to 1.7 times the average depth of the original section (note that it is assumed that the average depth before modification of the section is the same as the average depth after modification). The final modified section geometry is illustrated in Figure 11-18c.

Additional site conditions are as follows:

- flow conditions are gradually varying;
- channel characteristics are as described above;
- topographic survey indicates:
 - channel slope = 0.0024 ft/ft
 - channel width = 300 ft
 - bend radius = 1200 ft
- channel bottom is armored with cobble size material having a D_{50} of approximately 0.5 ft;
- bank soils are silty sand with the following soil characteristics:
 - $D_{85} = 0.0042$ ft.
 - $D_{50} = 0.0015$ ft.
 - $D_{15} = 0.00045$ ft
 - K (permeability) = 1.0×10^{-4} cm/s
- available rock riprap has a specific gravity of 2.60, and is described as angular;
- field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points;
- bank height along cut banks is approximately 9 ft.

Design Figures used to summarize data in this example are reproduced in Figures 11-20 and 11-22.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

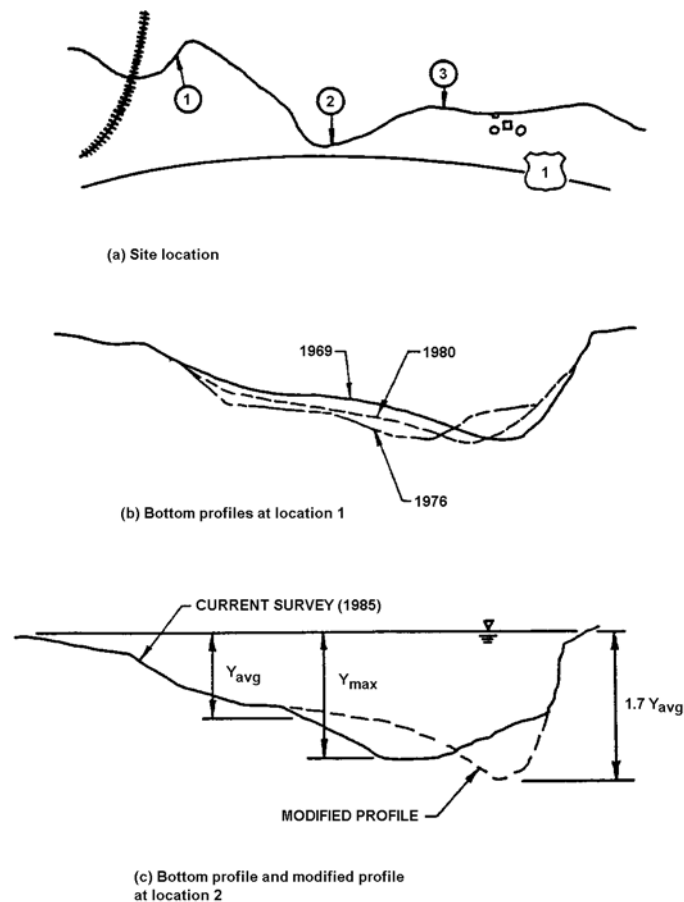


Figure 11-18 Channel Geometry Development (Example 2)

Step 1 Compile Field Data

- See given information for this example.
- See site history given above.

Step 2 Design Discharge

- Given as 46,700 ft³/s.
- From backwater analysis of this reach, it is determined that the discharge confined to the main channel (Q_{mc}) is 34,700 ft³/s.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 3 Design Cross Section

• Only the channel bank is to be stabilized; therefore, the channel section will consist of the existing channel with the bank graded to an appropriate angle to support the riprap revetment. Figure 11-18 illustrates the existing channel section.

- To minimize loss of bank vegetation, and limit the encroachment of the channel on adjacent lands, a 2H:1V bank slope is to be used.
- As given, the current bank height along the cut banks is 9 ft.

Step 4 Compute Design water Surface

- Determine roughness coefficient ($n = 0.042$).
This represents the average reach "n" used in the backwater analysis.
- Compute flow depth
 - Flow depth determined from backwater analysis. The maximum main channel depth was determined to be: $d_{\max} = 15$ ft.

Hydraulic radius for main channel

$R = 10.4$ ft (from backwater analysis)

R assumed (10 ft) is approximately equal to R actual, therefore, "n" as computed is OK.

Step 5 Determine Other Design Parameters

From backwater analysis: (all main channel values)

$$A = 2750 \text{ ft}^2$$

$$V_a = 12.6 \text{ ft/s}$$

$$d_a = d = 12.0 \text{ ft}$$

Step 6 Bank Angle Correction Factor

$$Q = 2:1$$

$$\Phi = 41^\circ$$

$$K_1 = 0.73 \text{ (from Figure 11-6)}$$

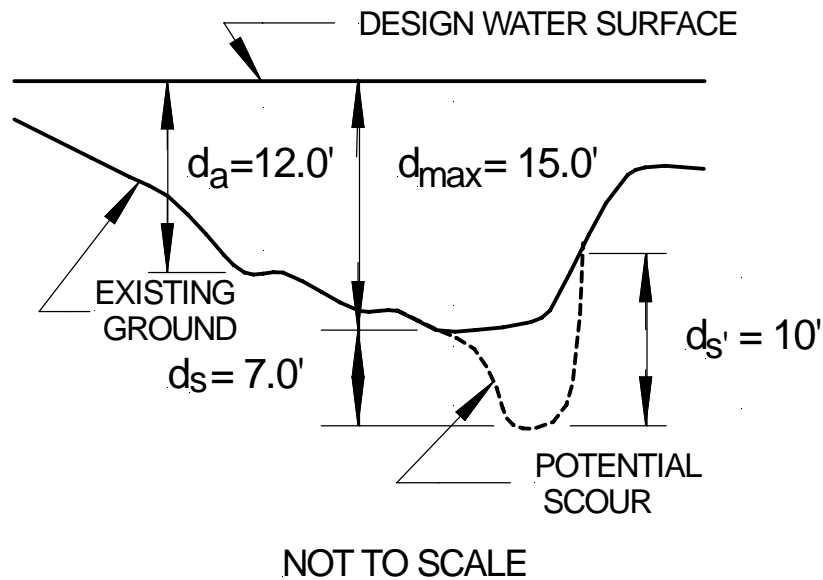
Step 7 Determine riprap size

- Using Figure 11-7
 $D_{50} = 0.9$ ft.
- Riprap specific gravity = 2.60 (given)

Stability factor = 1.6 (gradually varying flow, sharp bend — bend radius to width = 4)
 $C = 1.6$

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)



**Figure 11-19 Channel Cross Section For Example 2
Illustrating Flow And Potential Scour Depths**

(c) no piers or abutments to evaluate for this example, therefore
 $C_{p/a} = 1$

(d) Corrected riprap size

$$D'_{50} = D_{50}(1.6)(1.0) = 1.44 \text{ ft}$$

Step 8 Not applicable

Step 9 Select Design Riprap Size, Gradation and Layer Thickness (Preliminary Design of Waterway Area)

D_{50} size: Recommend AASHTO $\frac{1}{4}$ ton class riprap

$$D_{50} = 1.8 \text{ ft}$$

Gradation: See Figure 11-21

Layer thickness (T):

$$T = 2 D_{50} = 2(1.8) = 3.6 \text{ ft}, \quad \text{or } T = D_{100} = 2.25 \text{ ft}, \quad \text{Use } T = 3.6 \text{ ft}$$

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 10 Limits of Protection

(a) Longitudinal Extent of Protection

Field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points. Therefore, establish longitudinal limits of protection to extend to a point 300 ft (W) upstream of the bank entrance, and to a point 450 ft (1.5 W) downstream of the bend exit.

(b) Vertical Extent of Protection

Riprap entire channel bank from top-of-bank to below depth of anticipated scour. Scour depth evaluated as illustrated in section 11.6.7.2:

$$d_s = 6.5 D_{50}^{-0.11} \text{ (Equation 11.3)}$$

$$d_s = 6.5 (0.5)^{-0.11} = 7.0 \text{ ft.}$$

Adding this to the observed maximum depth yields a potential maximum scour depth of:

$$15.0 + 7.0 = 22.0 \text{ ft}$$

The bank material should be run to this depth, or a sufficient volume of stone should be placed at the bank toe to protect against the necessary depth of scour.

Step 11 Filter Layer Design

(a) Filter material size: (Figure 11-21)

$$5 < \frac{D_{15} \text{ [coarser layer]}}{D_{15} \text{ [finer layer]}} < 40$$

For the riprap to soil interface:

$$\frac{D_{15} \text{ [riprap]}}{D_{15} \text{ [soil]}} = \frac{0.5}{0.0042} = 119 > 5 \text{ and}$$

$$\frac{D_{15} \text{ [riprap]}}{D_{15} \text{ [soil]}} = \frac{0.5}{0.00045} = 1111 > 40$$

Therefore, a filter fabric is needed.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 12 Edge Details

(a) Flank details: See Figure 11-23

(b) Toe detail: See Figure 11-23

Anticipated scour depth below existing channel bottom at the bank (d'_s) is the depth of scour (computed in step 10) minus the current bed elevation at the bank (see Figure 11-21): $22 \text{ ft} - 12 \text{ ft} = 10 \text{ ft}$

Rock quantity required below the existing bed:

$$R_q = d'_s (\sin^{-1} \Theta) (T) (1.5) \quad (1.5)$$

Where: R_q = required riprap quantity per foot of bank, ft^2

Θ = the bank angle with the horizontal, degrees

T = the riprap layer thickness, ft

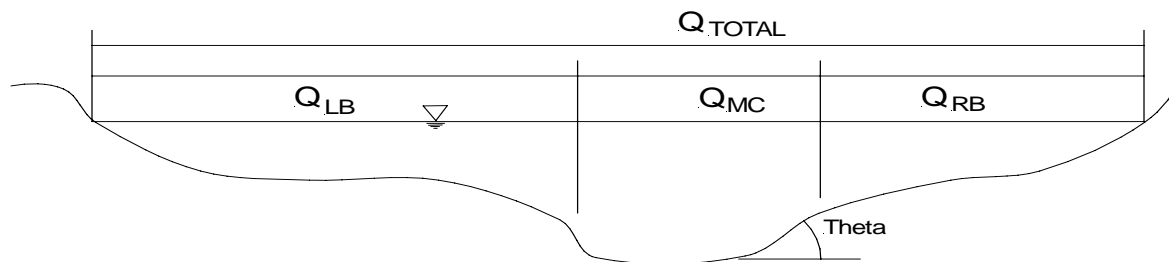
$$R_q = (10) (2.24) (3) (1.5) = 101 \text{ ft}^2$$

A 6 ft deep trapezoidal toe trench with side slopes of 2H: 1V and 1H: 1V, and a bottom width of 6 ft contains the necessary volume. Figure 11-23 illustrates the resulting toe trench detail.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Rock Riprap			
Project Name: _____		Project No.: _____	
Subject _____		Page ____ of ____	
By _____	Date _____	Checked By _____	Date _____



Discharge DATA			
Q_{total}	Q_{LB}	Q_{MC}	Q_{RB}

DATA SUMMARY	Bank	Bed
Area, Ft^2		
Velocity, (ft./sec.)		
Depth, d_a , ft		
Theta, θ		
Phi, ϕ		
K_1		
D_{50} , ft		
SF		
S_g , g		
C		
$C_{P/A}$		
D_{50}		

RIPRAP Characteristics		
SIZE		
D ₅₀		
Class		
THICKNESS		
2D ₅₀		
D ₁₀₀		
USE		
Gradation	Percent Finer	Size
	100	
	50	
	5-10	

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY >	

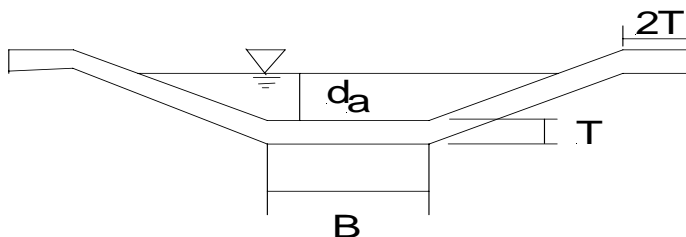


Figure 11-20 Riprap Size Form Example 2

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

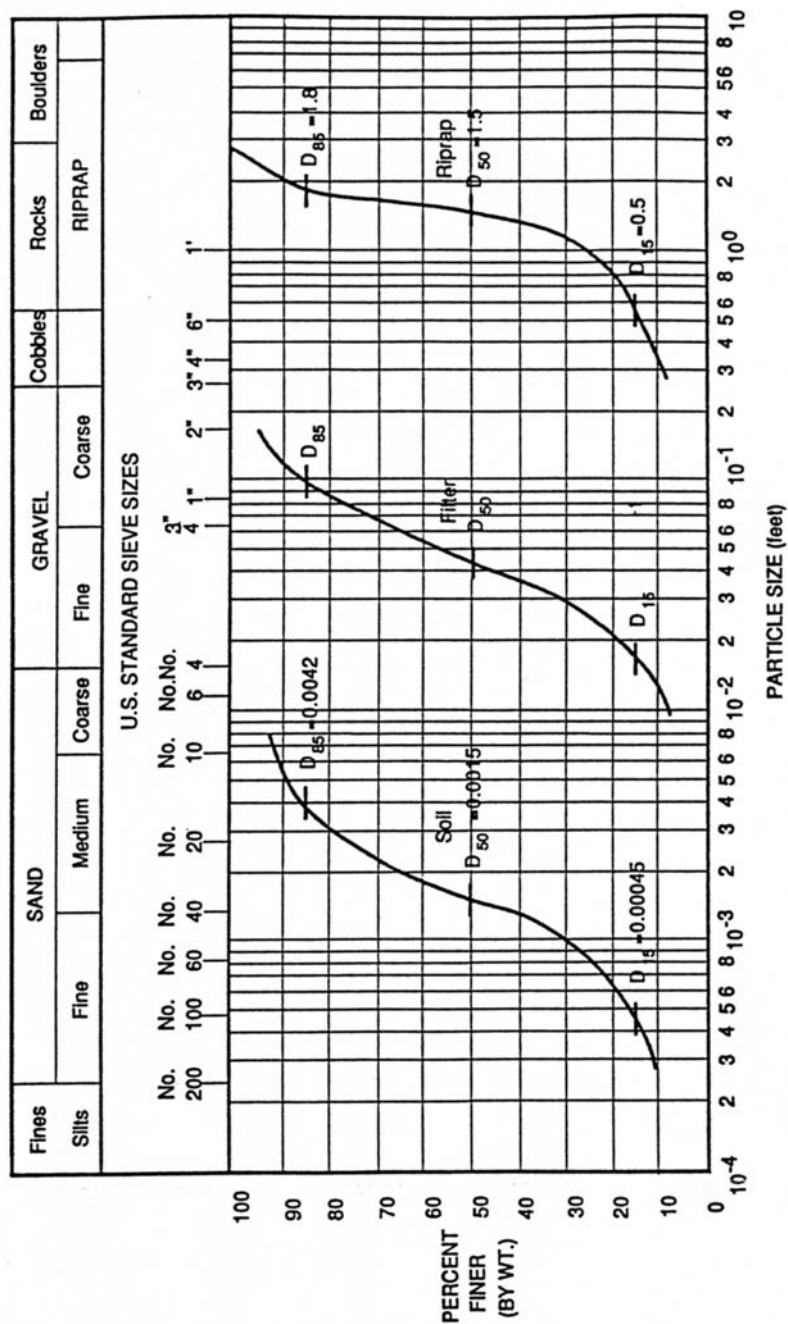


Figure 11-21 Material Gradation, Example 2 (Natural Soils)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

FILTER DESIGN			
Project Name: _____		Project No.: _____	
Subject _____		Page ____ of ____	
By _____	Date _____	Checked By _____	Date _____

GRANULAR FILTER

LAYER	DESCRIPTION	D ₁₅	D ₈₅	RATIO:			
				$\frac{D_{15} \text{ RIPRAP}}{D_{85} \text{ SOIL}}$	<5	$\frac{D_{15} \text{ RIPRAP}}{D_{15} \text{ SOIL}}$	<40
	Riprap	0.60					
	Soil	0.0045	0.105				
				6	No		

DATA SUMMARY

LAYER DESCRIPTION	D ₁₅	D ₈₅	THICKNESS
Riprap	0.60		
Soil	0.0045	0.105	

FABRIC FILTER

PHYSICAL PROPERTIES CLASS _____

HYDRAULIC PROPERTIES:

PIPING RESISTANCE, 50% PASSING #200 AOS<0.6mm

PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: _____

Figure 11-22 Filter Design, Example 2

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

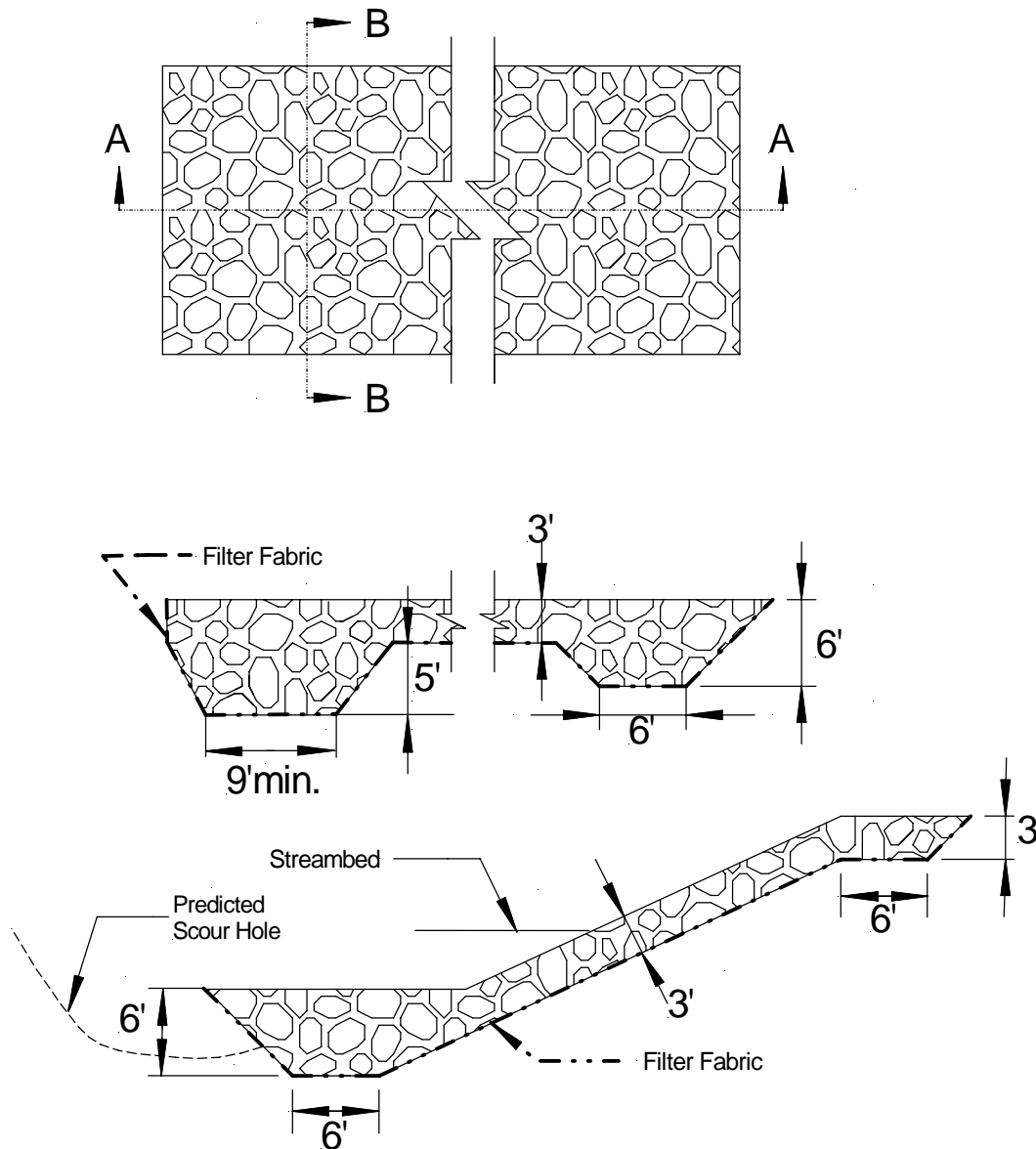


Figure 11-23 Toe And Flank Details (Example 2)

11.7 Design Guidelines (continued)

11.7.2 Wire-Enclosed Rock

As described in Section 11.5.1, wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks.

The wire cages that make up the mattresses and gabions are available from commercial manufacturers. If desired, the wire baskets can also be fabricated from available wire fencing materials.

Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress. They are commonly used on flatter slopes, less than 2:1, or as aprons. Commercial sizes are usually 6, 9 or 12 inches in thickness and 6, 9 or 12 feet wide.

Stacked block gabion revetments consist of rectangular wire baskets that are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls that help to support other upper bank revetments and prevent undermining. The rectangular basket or gabion units used for stacked configurations are of more uniform dimensions than those typically used for mattress designs. That is, they typically have a square cross section. Commercially available gabions used in stacked configurations are available in various sizes but the most common have a 3 ft width and thickness.

11.7.2.1 Design Guidelines For Mattresses

Components of a rock and wire mattress design include layout of a general scheme or concept, bank and foundation preparation, mattress size and configuration, stone size, stone quality, basket or rock enclosure fabrication, edge treatment and filter design. Design guidance is provided below in each of these areas.

General

Rock and wire mattress revetments can be constructed from commercially available wire units or from available wire fencing material. The basic elements of a mattress is illustrated in Figure 11-24. Rock and wire mattress revetments can be used to protect either the channel bank, Figure 11-25, or the entire channel perimeter, Figure 11-26.

When used for bank protection, Figure 11-25, rock and wire mattress revetments consist of two distinct sections: a toe section and upper bank paving. As illustrated, a variety of toe designs can be used. The vertical and longitudinal extent of the mattress should be based on guidelines provided in section 11.6.7. Emphasis in design should be placed on toe design, and filter design. These designs are detailed later.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the rock and wire mattress is to be constructed, should not deviate from the specified slope line by more than 6 inches. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed.

Mattress Unit Size and Configuration

Individual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes as indicated in Table 11-6. Manufacturer's literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. It is recommended that diaphragms be installed at a nominal of 3 ft spacing within each of the gabion units to provide the recommended compartmentalization, Figure 11-24. Wire mattress units should be limited to slopes no steeper than 2:1.

Table 11-6 Standard Gabion Sizes

Thickness (ft)	Width (ft)	Length (ft)	Wire-Mesh Opening Size (in. × in.)
0.75	6	9	2.5 × 3.25
0.75	6	12	2.5 × 3.25
1.0	3	6	3.25 × 4.5
1.0	3	9	3.25 × 4.5
1.0	3	12	3.25 × 4.5
1.5	3	6	3.25 × 4.5
1.5	3	9	3.25 × 4.5
1.5	3	12	3.25 × 4.5
3.0	3	6	3.25 × 4.5
3.0	3	9	3.25 × 4.5
3.0	3	12	3.25 × 4.5

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

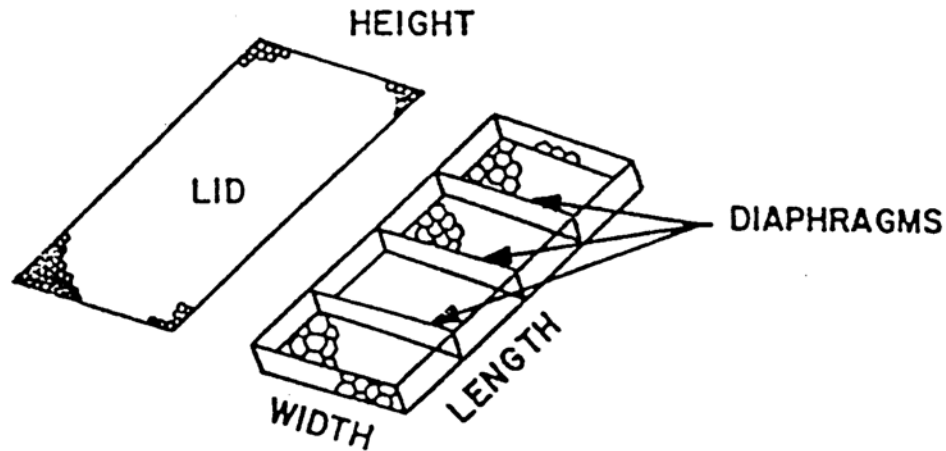


Figure 11-24 Mattress Configuration

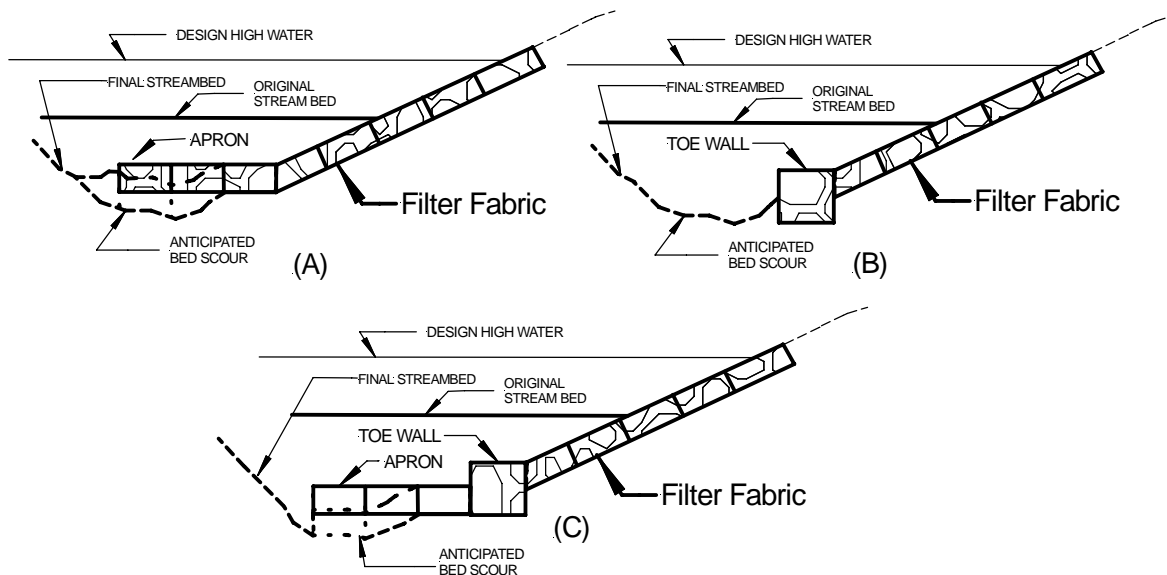


Figure 11-25 Rock And Wire Mattress Configuration: (a) mattress with toe apron; (b) mattress with toe wall; and (c) mattress with toe wall and apron

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

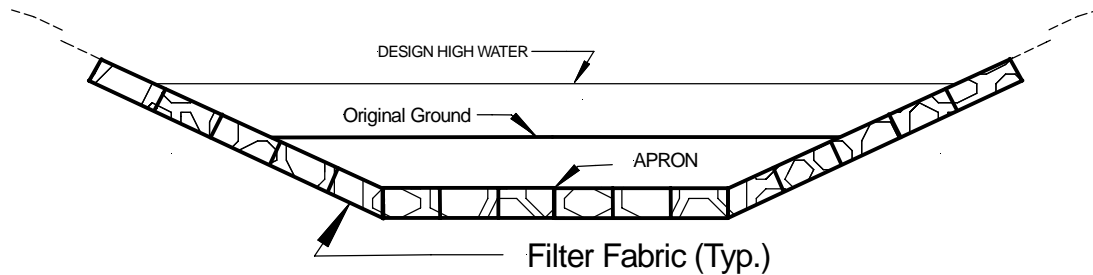


Figure 11-26 Wire-Tied Rock Mattress Installation Covering The Entire Channel Perimeter

The thickness of the mattress is determined by three factors: the erodibility of the bank soil, the maximum velocity of the water and the bank slope. The minimum thickness required for various conditions is tabulated in Table 11-7. These values are based on observations of a large number of mattress installations which assume a filling material in the size range of 3 to 6 inches. The mattress thickness should be at least as thick as two overlapping layers of stone. The thickness of mattresses used as bank toe aprons should always exceed 12 inches. The typical range is 12 to 15 inches. The thickness of mattress revetments can vary according to need by utilizing gabions of different depths.

Table 11-7 Criteria For Mattress/Gabion Thickness

Bank Soil Type	Maximum Velocity (ft/s)	Bank Slope (H:V)	Min. Required Mattress Thickness (inches)
Cohesive Soils	10	< 3:1	9
	13-16	< 2:1	12
Silts, fine sands	10	< 2:1	12
Shingle with gravel	16	< 3:1	9
	20	< 2:1	12

For any installation on a slope greater than 2:1 the minimum mattress thickness shall be 18 inches.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

Stone Size

The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available, and 95% of the stone, by weight, should be slightly larger than the wire-mesh opening. For commercially available units, the wire-mesh opening sizes are listed in Table 11-6.

Common median stone sizes used in mattress designs range from 3 to 6 inches for mattress less than 12 inches thick. For mattresses of larger thickness, rock having a median size up to 12 inches is used.

Stone Quality

The stone should meet the quality requirements as specified in Section 913 of the ADOT standard specifications.

Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire is approximately No. 12 gage. The wire for edges and corners is approximately 12 gage. Manufacturer's instructions for field assembly of basket units should be followed.

All wire used in the construction of the mesh rock enclosures including tie wire shall be galvanized in accordance with the ADOT Standard Specifications, Section 913.

Galvanized wire baskets may be safely used in fresh water and in areas where the pH of the liquid in contact with it is not greater than 10. For highly corrosive conditions such as industrial areas, polluted streams and in soils such as muck, peat and cinders, a polyvinyl chloride (PVC) coating must be used over the galvanizing. The PVC coating must have a nominal thickness of 0.02165 inches and shall nowhere be less than 0.015 inches. It shall be capable of resisting deleterious effects of natural weather exposure and immersion in salt water, and shall not show any material difference in its initial characteristics with time.

Edge Treatment

The edges of rock and wire mattress revetment installations (the toe, head and flanks) require special treatment to prevent damage from undermining. Of primary concern is toe treatment. Figure 11-25 illustrates several possible toe configurations. If a toe apron is used, its projection should be 1.5 times the expected maximum depth of scour in the vicinity of the revetment toe. In areas where little toe scour is expected, the apron can be replaced by a single-course gabion toe wall which helps to support the revetment and prevent undermining. In cases where an excessive amount of toe scour is anticipated, both an apron and a toe wall can be used.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

To provide extra strength at the revetment flanks, it is recommended that mattress units having additional thickness be used at the upstream and downstream edges of the revetment, Figure 11-28. It is further recommended that a thin layer of topsoil be spread over the flank units to form a soil layer to be seeded when the revetment installation is complete.

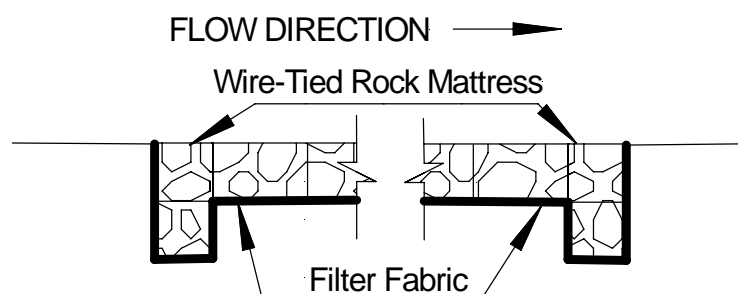
Filter Design

Individual mattress units will act as a crude filter as well as a pavement unit when filled with overlapping layers of hand-size stones. However, it is recommended that a layer of permeable membrane cloth (geotextile) woven from synthetic fibers should be placed between the bank and the rock and wire mattress revetment to inhibit washout of fines.

Construction

Construction details for rock and wire mattresses vary with the design and purpose for which the protection is provided. Rock and wire mattress revetments may be fabricated where they are to be placed, or at an off-site location. Fabrication at an off-site location requires that the individual mattress units be transported to the site. In this case extreme care must be taken so that moving and placing the baskets does not damage them by breaking or loosening strands of wire or ties, or by removing any of the galvanizing or PVC coating. Because of the potential for damage to the wire enclosures, off-site fabrication is not recommended.

On-site fabrication of rock and wire mattress revetments is the most common practice. Figure 11-25 illustrates installations on a channel bank. Figure 11-26 illustrates an installation where the entire channel perimeter is lined. Installation of mattress units above the water line is usually accomplished by placing individual units on the prepared bank, lacing them together, filling them with appropriately sized rock and then lacing the tops to the individual units.



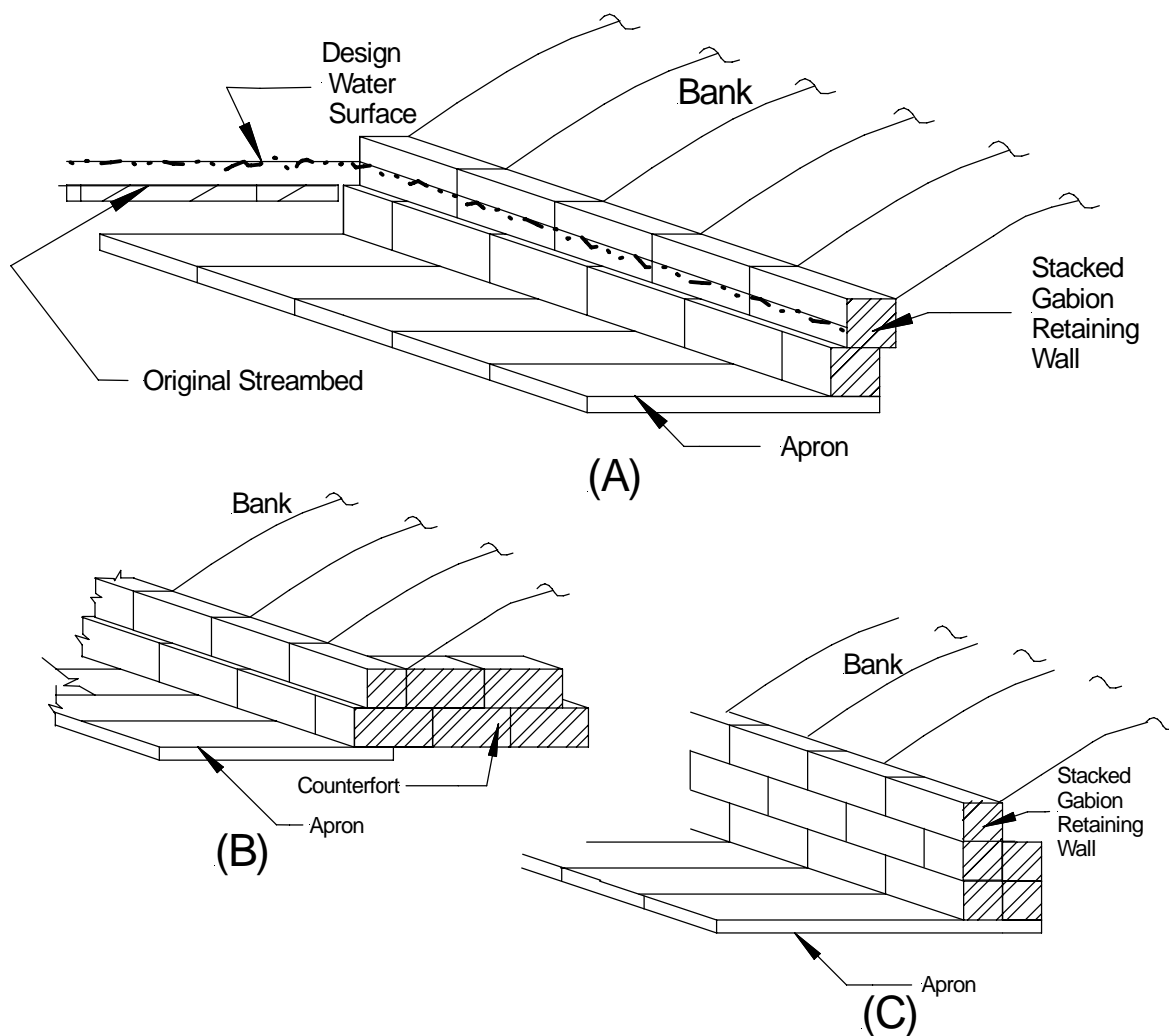
**Figure 11-27 Flank Treatment for Wire-Tied Rock and Mattress Designs
-Upstream Face and Downstream Face**

11.7 Design Guidelines (continued)

11.7.2 Wire-Enclosed Rock (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions

Components of stacked gabion revetment design include layout of a general scheme or concept, bank and foundation preparation, unit size and configuration, stone size and quality, edge treatment, backfill and filter considerations and basket or rock enclosure fabrication. Design guidelines for stone size and quality, and bank preparation are the same as those discussed for mattress designs, other remaining areas are discussed below.



**Figure 11-28 Typical Stacked Block Gabion Revetment Details:
stepped back low retaining wall with apron.**

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

General

Stacked gabion revetments are typically used instead of gabion mattress designs when the slope to be protected is greater than 3H:1V or when the purpose of the revetment is for flow training. They can also be used as retaining structures when space limitations prohibit bank grading to a slope suitable for other revetments.

Stacked gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below any anticipated scour depth. Additionally, in alluvial streams where channel bed fluctuations are common, an apron should be used as illustrated in Figures 11-25. Aprons are also recommended for situations where the estimated scour depth is uncertain.

Size and Configuration

Common commercial sizes for stacked gabions are listed in Table 11-6. The most common sizes have widths and depths of 3 ft. Sizes less than 1 ft thick are not practical for stacked gabion installations.

Retaining walls can be designed in either a stepped-back configuration as illustrated in Figures 11-28(a) or a batter configuration as illustrated in Figure 11-28(c). Structural details and configurations can vary from site to site.

Gabion walls are gravity structures and their design follows standard engineering practice for retaining structures. Design procedures are available in standard soil mechanics texts as well as in gabion manufacturer's literature.

Edge Treatment

The flanks and toe of stacked block gabion revetments require special attention. The upstream and downstream flanks of these revetments should include counterforts, see Figure 11-28(b). The counterforts should be placed 12 to 18 ft from the upstream and downstream limits of the structure, and should extend a minimum of 12 ft into the bank.

Backfill/Filter Requirements

Standard retaining wall design requires the use of selected backfill behind the retaining structure to provide for drainage of the soil mass behind the wall. The permeable nature of gabion structures permits natural drainage of the supported embankment. However, since material leaching through the gabion wall can become trapped and cause plugging, it is recommended that a geotextile filter be used, Figure 11-25.

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

The toe of the revetment should be protected by placing the base of the gabion wall at a depth below anticipated scour depths. In areas where it is difficult to predict the depth of expected scour, or where channel bed fluctuations are common, it is recommended that a mattress apron be used. The minimum apron length should be equal to 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour depth.

Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire specifications are the same as those discussed for mattress units. Specifications for galvanizing and PVC coatings are also the same for block designs as for mattresses. Figure 11-29 illustrates typical details of basket fabrication.

Construction

Construction details for gabion installations typically vary with the design and purpose for which the protection is being provided. Several typical design schematics were presented in Figures 11-28. Design details for a typical stepped-back design and a typical batter design are presented in Figure 11-30.

As with mattress designs, fabrication and filling of individual basket units can be done at the site, or at an off-site location. The most common practice is to fabricate and fill individual gabions at the design site. The following steps outline the typical sequence used for installing a stacked gabion revetment or wall:

Step 1 Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

Step 2 Place the filter and gabion mattress (for designs which incorporate this component) on the prepared grade, then sequentially stack the gabion baskets to form the revetment system.

Step 3 Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

Step 4 Fill the gabions to a depth of 1 ft with stone from 6 to 12 inches in diameter. Place one connecting wire in each direction and loop it around two meshes of the gabion wall. Repeat this operation until the gabion is filled.

Step 5 Wire adjoining gabions together by their vertical edges; stack empty gabions on the filled gabions and wire them at front and back.

Step 6 After the gabion is filled, fold the top shut and wire it to the ends, sides and diaphragms.

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

Step 7 Crushed stone and granular backfill should be placed at intervals to help support the wall structure. It is recommended that backfill be placed at three-course intervals.

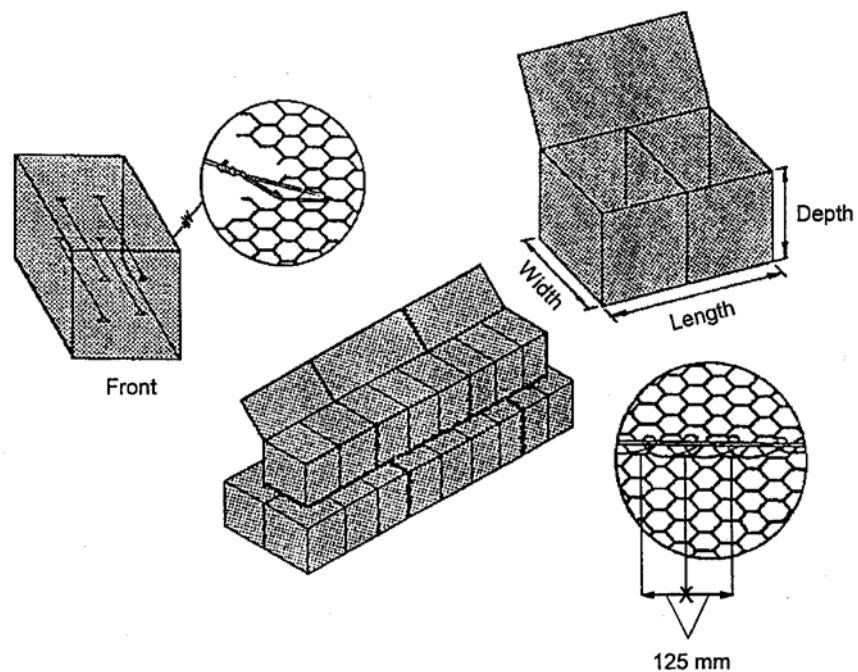
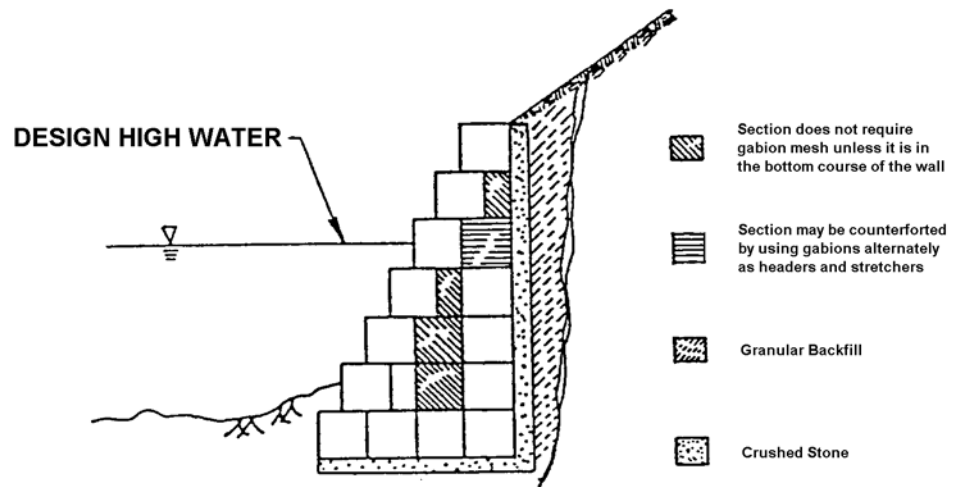


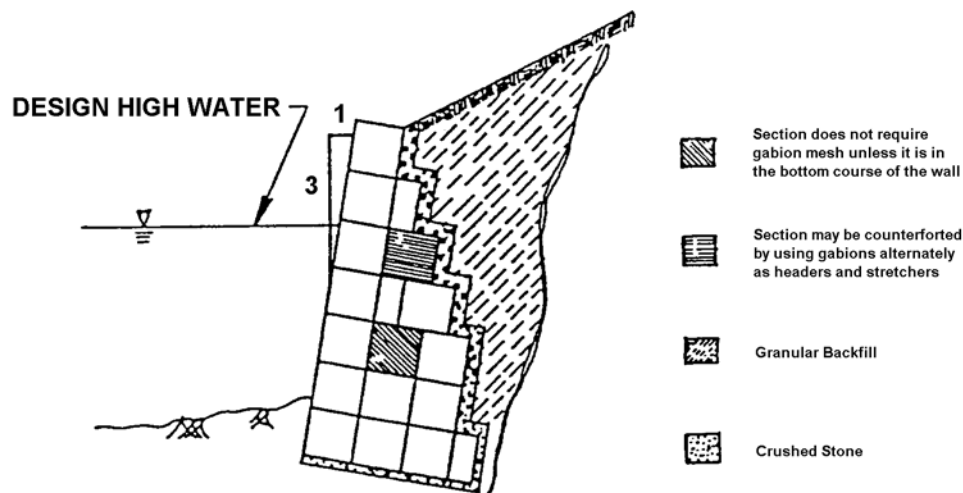
Figure 11-29 Gabion Basket Fabrication

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)



(a)



(b)

Figure 11-30 Section Details: (a) stepped back and (b) battered gabion retaining walls

11.7 Design Guidelines (continued)

11.7.3 Soil-Cement

Soil-cement is an acceptable method of slope protection for dikes, levees, channels, and highway embankments. Soil-cement can also be used to construct impervious cores as well as provide a protective facing. On most projects, soil-cement is constructed in stair-step fashion by placing and compacting the soil-cement in horizontal layers stair-stepped up the embankment (Figure 11-31). A compacted layer thickness of 6 inches is most widely used, with the recommended maximum being 9 inches for efficient, uniform compaction. This facilitates placement using common highway construction equipment. Embankment slopes of 1:1 to 2:1 are most common for stair-step construction, slopes steeper than 2:1 may need to be evaluated for stability. The width of a layer may need to be adjusted to provide a **minimum protective thickness of about 3.5 ft.** measured normal to the slope.

A wide variety of soils can be used to make durable soil-cement slope protection. The Portland Cement Association (PCA) has data on soil types, gradations, costs and testing procedures. The PCA also has data placement and compaction methods.

Use of soil-cement does not require any unusual design considerations for the embankment. Proper embankment design procedures should be followed, based on individual project conditions, to prevent subsidence or any other type of embankment distress.

11.7.3.1 Design Guidelines

Top, Toe and End Features

An important consideration in the design of soil-cement facing is to ensure that all extremities of the facing are tied into non-erodible sections. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 11-31, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly into the bank and be tied into the bank with counterforts.

Where miscellaneous structures such as culverts extend through the facing, the area immediately adjacent to such structures are constructed by placing and compacting the soil-cement by hand or with small power tools, or by using a lean-mix concrete.

Special Conditions

Slope stability is provided to embankments by the strength and impermeability of the soil-cement facing. Special design considerations usually are not necessary in soil-cement-faced embankments. It is necessary to utilize proper design and analysis procedures to ensure the structural and hydraulic integrity of the embankment. Conditions most commonly requiring special analysis include subsidence of the embankment or rapid drawdown of the reservoir or river.

11.7 Design Guidelines (continued)

11.7.3 Design Guidelines (continued)

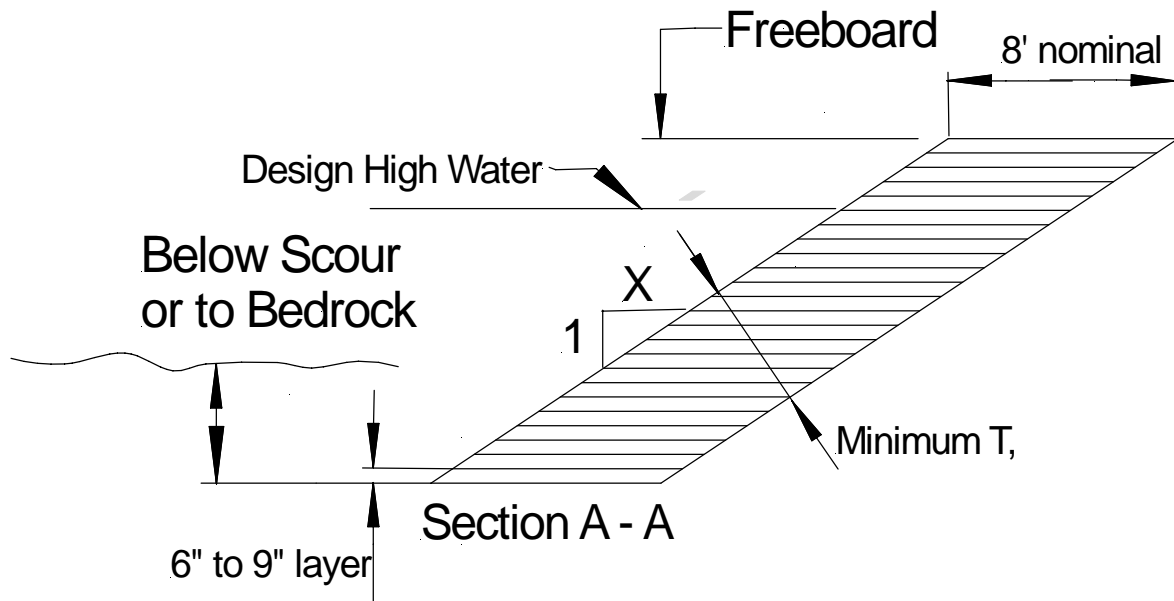


Figure 11-31 Details And Dimensions Of Three Soil-Cement Facings Design Guidelines

Subsidence

Embankment subsidence results from a compressible foundation, settlement within the embankment itself, or both. Analyzing the possible effects of such a condition involves a number of assumptions by the designer concerning the embankment behavior. Combining these assumptions with the characteristics of the facing, a structural analysis of the condition can be made. The layer effect can usually be ignored.

Note: The post construction appearance of a pattern of narrow surface cracks about 10 to 20 ft apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence conceivably could allow the facing to settle back in large sections coinciding with the normal shrinkage crack pattern. If such settlement of the soil-cement, with separation at the shrinkable cracks, takes place, the slope remains adequately protected unless the settlement is large enough to allow the outer face of a settling section to move past the inner face of an adjoining section.

11.7 Design Guidelines (continued)

11.7.3 Design Guidelines (continued)

Rapid Drawdown

Rapid drawdown exceeding 15 ft or more within a few days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. Three design concepts that may be used to prevent damage due to rapid drawdown-induced pressure are:

1. Designing the embankment so that its least permeable zone is immediately adjacent to the soil-cement facing, which ensures that seepage through cracks in the facing will not build up a pool of water sufficient to produce damaging hydrostatic pressure;
2. evaluating the stability of the soil cement mass using gravity wall approach; and
3. providing free drainage behind, through or under the soil-cement facing to prevent adverse hydrostatic pressure.

11.7.3.2 Construction

The method of construction (central plant or mixed in place) should be considered by the designer in determining the facing cross section. Both methods have been successfully used for soil-cement slope protection. The central plant method allows faster production and provides maximum control of mixing operations. With the mixed-in-place method, mixing should be deep enough so that there will be no unmixed seams between the layers, but excessive striking of the soil-cement below the layer being mixed should be avoided. The PCA has sample specifications regarding these two construction methods.

11.7.4 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. See section 11.5.4 for additional descriptive information and general performance characteristics for grouted rock.

11.7.4.1 Design Guidelines

Components of grouted rock riprap design include layout of a general scheme or concept, bank preparation, bank slope, rock size and blanket thickness, rock grading, rock quality, grout quality, edge treatment, filter design and pressure relief. Grouted riprap designs are rigid monolithic bank protection schemes. When complete they form a continuous surface. A typical grouted riprap section is shown in Figure 11-32. Grouted riprap should extend from below the anticipated channel bed scour depth to the design high water level, plus additional height for freeboard. During the design phase for a grouted riprap revetment, special attention needs to be paid to edge treatment, foundation design and mechanisms for hydrostatic pressure relief.

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

Bank And Foundation Preparation

In general, the graded surface should not deviate from the specified slope line by more than 6 inches. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions. Since grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain soil permeability similar to that of the natural, undisturbed bank material.

Bank Slope

Bank slopes for grouted riprap revetments should not exceed 1.5:1.

Rock Size And Blanket Thickness

Grouted riprap is usually used because larger sizes of dumped riprap are not available. Therefore the largest size of economically available rock should be used. Blanket thickness and rock size requirements for grouted riprap installation are interrelated. The blanket thickness should be a minimum of 150% of the median rock size. The largest rock used in the revetment should not exceed the blanket thickness. Figure 11-37 illustrates a relationship between the design velocity and the minimum required riprap blanket thickness for grouted riprap designs.

Rock Grading

Table 11-9 provides guidelines for rock gradation in grouted riprap installations. Five size classes are listed.

Rock Quality

Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

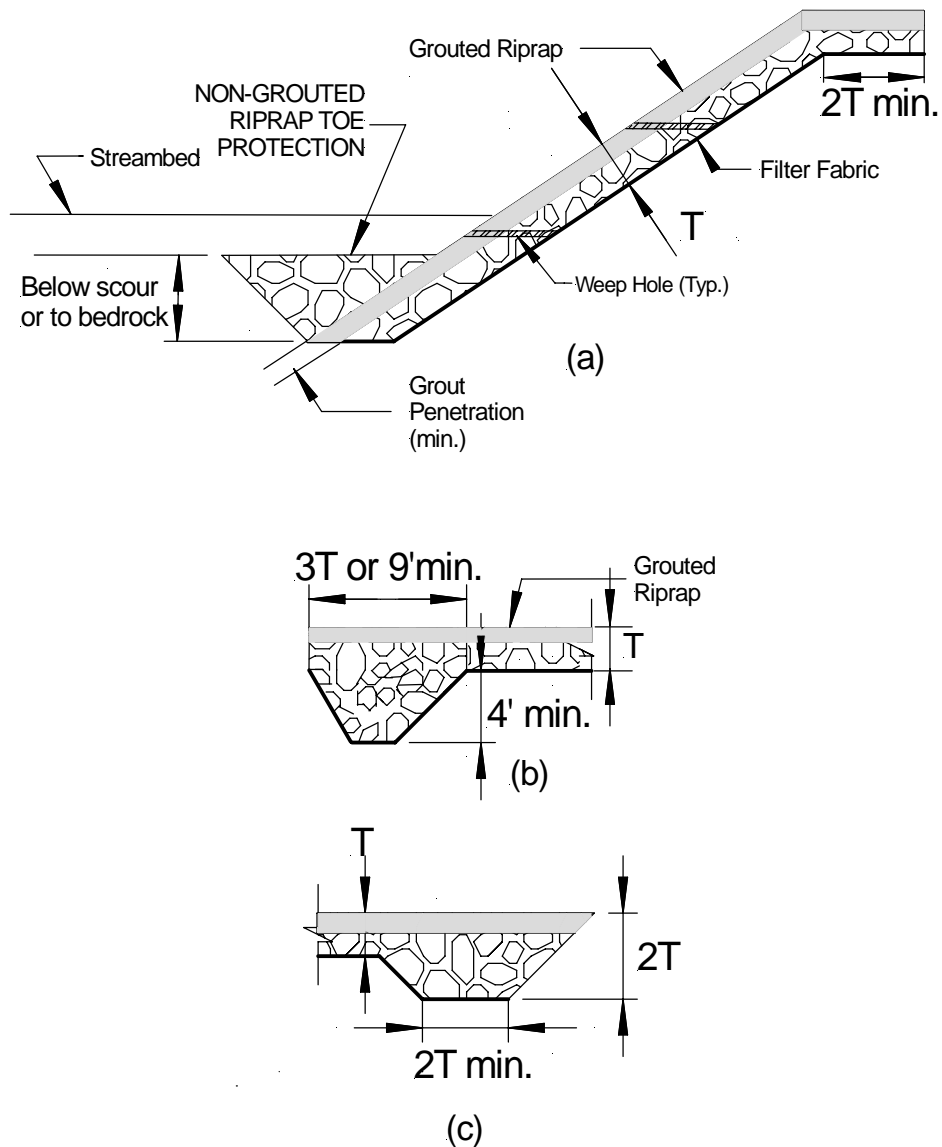


Figure 11-32 Grouted Riprap Sections: (a) section; (b) Upstream Detail; and (c) Downstream Detail

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

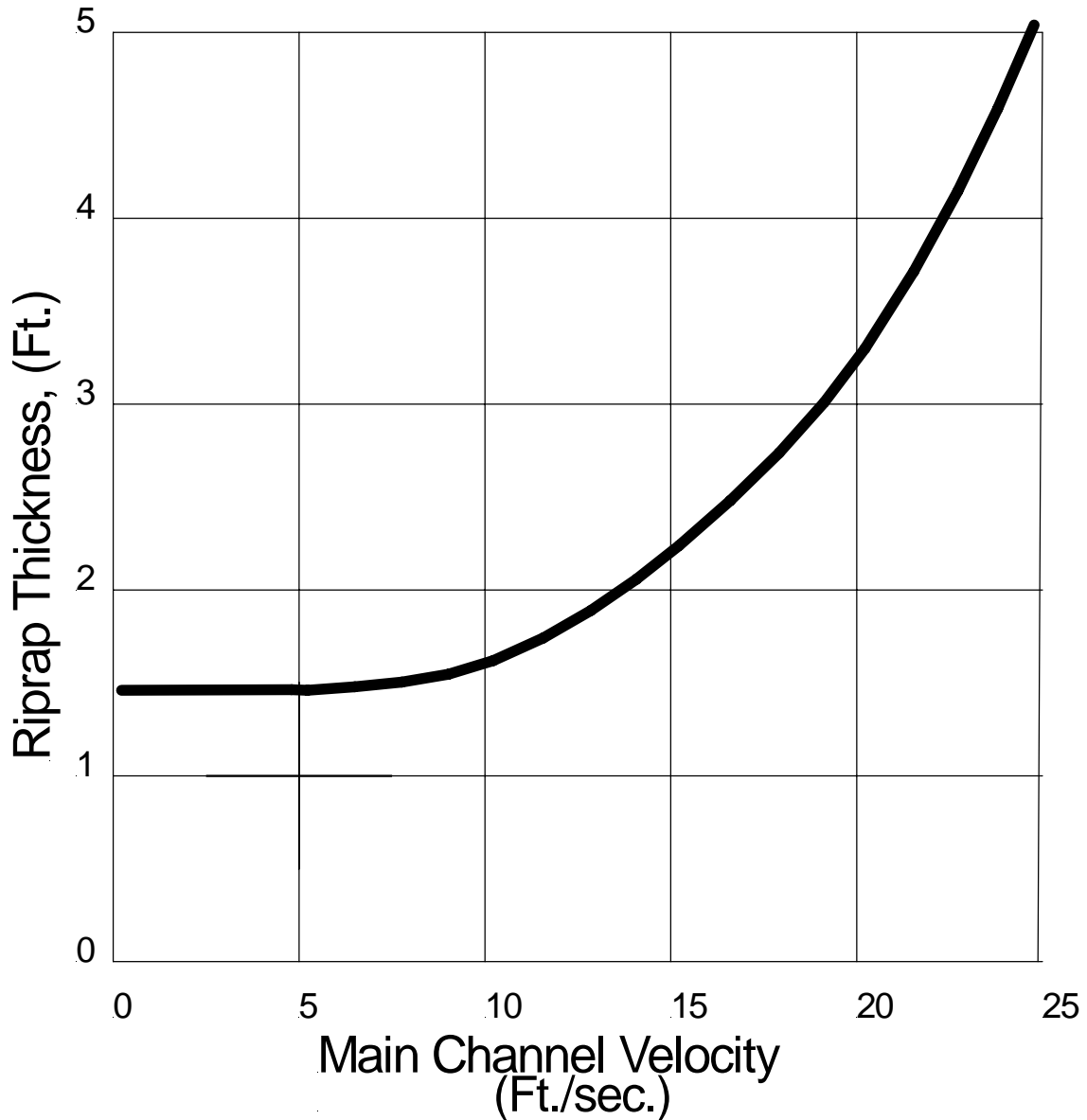


Figure 11-33 Required Blanket Thickness As A Function Of Flow Velocity

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

Grout Quality And Characteristics

Grout should consist of good strength concrete using a maximum aggregate size of $\frac{3}{4}$ in and a slump of 3 to 4 inches. Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability. The thickness of grout necessary is shown in Table 11-9. The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate.

Edge Treatment

The edges of grouted rock revetments (the head, toe and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 11-32(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with grout-free riprap. The grout-free riprap provides extra protection against undermining at the bank toe. To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in Figure 11-32, (b) and (c).

Filter Design

Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-inch granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface, Figure 11-32(a). Weeps should extend through the grout surface to the interface with the gravel under-drain layer. Weeps should consist of 3-inch diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

11.7 Design Guidelines (continued)

11.7.4.2 Construction

The following construction procedures are recommended:

Step 1 Normal construction procedures include (a) bank clearing and grading; (b) development of foundations; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) backfilling toe and flank trenches; and (f) vegetation of disturbed areas.

Step 2 The rock should be wet immediately prior to commencing the grouting operation.

Step 3 The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment, or any other mechanical method which will control segregation and uniformity of the grout.

Step 4 Spading and rodding are necessary where penetration is achieved by gravity flow into the interstices.

Step 5 No loads should be allowed upon the revetment until good strength has been developed

Table 11-9 Recommended Grading Of Grouted Rock Slope Protection

Rock sizes	Classes					
	(Per cent larger than given rock size)					
Equivalent Diameter, Ft.	Weight, Ton	1 Ton	½ Ton	¼ Ton	Light	Facing
3.5	2	0-5				
2.75	2	50-100	0-5			
2.25	0.5	---	50-100	0-5		
1.75	0.25	95-100	---	50-100	0-5	
1.25	0.1	---	95-100	---	50-100	0-5
1	0.0375	---	---	95-100	95-100	50-100
0.5	0.0125	---	---	---	---	95-100
Minimum Penetration of Grout (inches)		24	18	14	10	8

11.7 Design Guidelines (continued)

11.7.5 Concrete Slope Pavement

Concrete slope pavement revetments are cast-in-place, or precast and set in place on a prepared slope to provide a continuous, monolithic armor for bank protection. Cast-in-place designs are the most common of the two design methods. For additional descriptive information and general performance characteristics of concrete pavement see Section 11.5.3.

11.7.5.1 Design Guidelines

Components of concrete pavement revetment design include layout of a general scheme, bank and foundation preparation, bank slope, pavement thickness, pavement reinforcement, edge treatment, stub walls, filter design, pressure relief and concrete quality. Each of these components is addressed below.

Concrete pavement designs are rigid monolithic bank protection schemes. When complete they form a continuous surface. As illustrated in Figure 11-34, typical concrete pavement revetment consists of the bank pavement, a toe section, a head section, cutoff or stub walls, weeps and a filter layer. The various dimensions are labeled in Table 11-10.

As indicated in Figure 11-34, concrete pavements should extend vertically below the anticipated channel bed scour depth, and to a height equal to the design high water level plus additional height for freeboard. The longitudinal extent of protection should be as described in section 11.7.6. One additional consideration in concrete pavement design is the surface texture. Depending on the smoothness required for hydraulics, a float or sand finish may be specified, or if roughness is desired, plans may call for a deformed surface obtained by raking the surface after the initial set.

During the design phase for concrete pavement revetment, special attention needs to be paid to toe and edge treatment, foundation design and mechanisms for hydrostatic pressure relief. Field experience indicates that inadequacies in these areas of design are often responsible for failures of concrete pavement revetments.

Table 11-10 Dimensions For Concrete Slab

<u>Dimension</u>											
A	B	C	D	E	F	G	H	I	J	K	L
6	9	9	1' 9	2' 0	9	6	4-5	2'-3'	1' 6	25'-30'	9

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

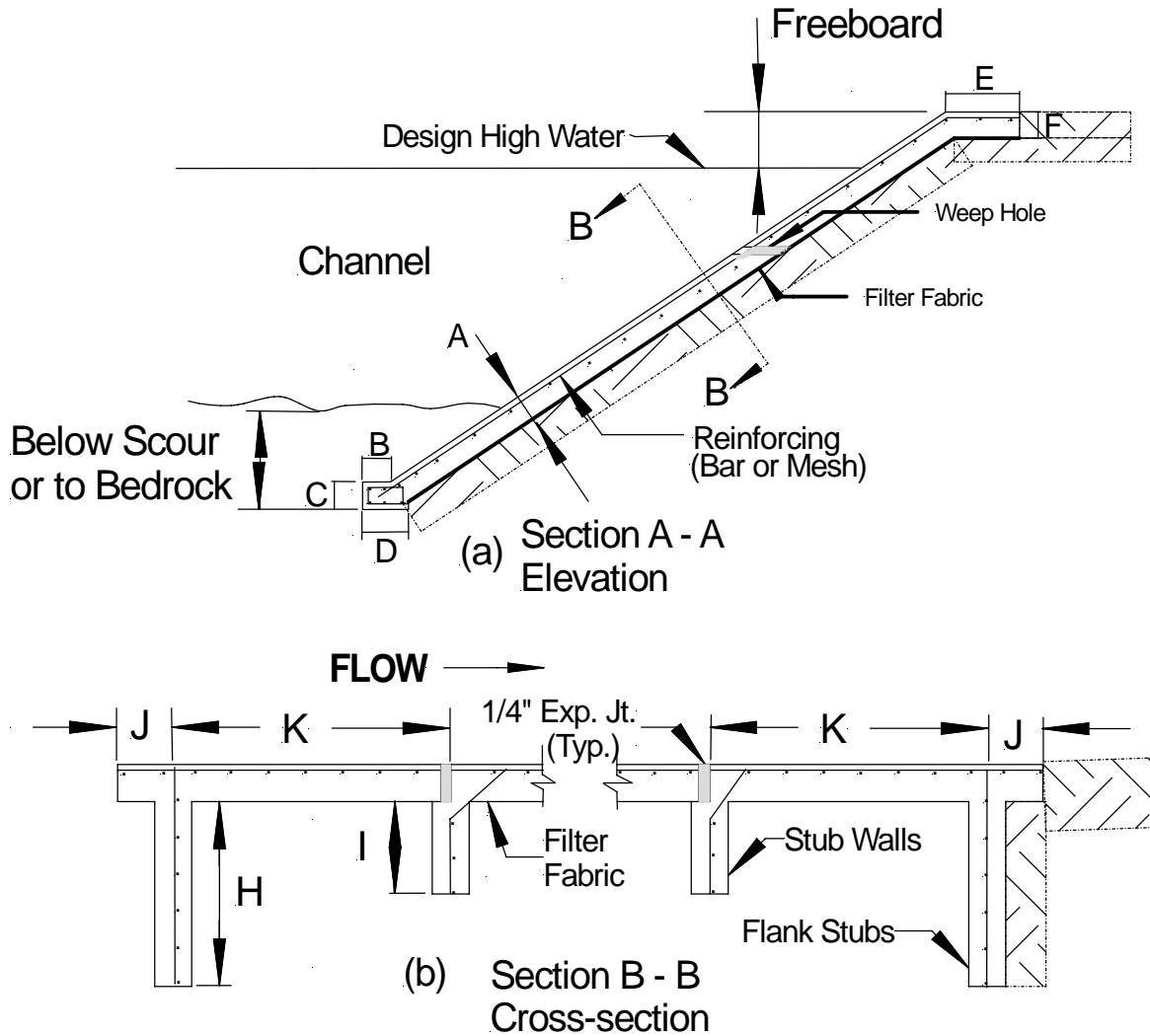


Figure 11-34 Concrete Slope Paving Detail:

(a) typical cross-section, section A-A;

(b) typical longitudinal section, section B-B

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

Bank And Foundation Preparation

The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to a slope not to exceed 1.5:1. Continuity of the final graded surface is important. After grading, the surface should be true to grade, and stable with respect to slip and settlement. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain a soil permeability similar to that of the natural, undisturbed bank material. After compaction, the bank surface should not deviate from the specified slope by more than several centimeters at any one point. This is particularly true if pre-cast slabs are to be placed on the bank.

The foundation for the concrete slope pavement revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of water in the wedge above the revetment for design conditions, whichever is greater.

Bank Slope

The bank slope for concrete pavements should not exceed 1.5:1.

Pavement Thickness

A minimum pavement thickness of 6 inches is recommended. Dimensions are provided in Table 11-10 for Figure 11-34.

Reinforcement

The purpose of reinforcement is to maintain the continuity of pavement by aggregate interlock even though cracks develop from shrinkage, thermal stresses and flexural stresses. Reinforcement may be either mesh or bar reinforcement. Typically, #5 rebars are used in 6-inch slabs. Both size and spacing in each direction must be specified.

Concrete Quality

Concrete should be of good strength, and the concrete mixture shall be proportioned so as to secure a workable, finishable, durable, watertight and wear resistant concrete of the desired strength.

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

Edge Treatment

The edges of the concrete pavement (the toe, head and flanks) require special treatment to prevent undermining. Section A-A in Figure 11-34 illustrates standard head and toe designs. The head of the pavement should be tied into the bank and overlapped with soil as illustrated to form a smooth transition from the concrete pavement to the natural bank material. This minimizes scour due to the discontinuity in this area. Also, this design seals off the filter layer from any water that overtops the revetment, thereby reducing the potential for erosion at this interface. Section A-A also illustrates the standard toe design. The revetment toe should extend to a depth below anticipated scour or to bedrock. When this is not feasible without costly underwater construction, an alternative design should be considered. Several alternative designs are illustrated in Figure 11-35, including a riprap filled toe trench, a toe mattress and a sheet-pile toe wall. In all but the latter case, the concrete pavement should extend a minimum of 5 ft below the channel thalweg; the sheet-pile toe wall can be attached to the concrete pavement above, below, or at the channel bed level.

Section B-B of Figure 11-34 illustrates flank treatment. At the upstream and downstream blanks, flank stubs are used to prevent progressive undermining at the flanks.

Stub Walls

As illustrated in Figure 11-34, stub walls should be placed at regular intervals. Stub walls provide support for the revetment at expansion joints; they also guard against progressive failure of the revetment. A maximum spacing of 25 feet is suggested.

Filter Design

Filters are required under all concrete pavement revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 4- to 6-inch granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly graded granular material underlain with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

11.7 Design Guidelines (continued)

11.7.5 Design Guidelines (continued)

Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the pavement surface (Figure 11-34). Seeps should extend through the pavement surface and into the granular underdrain or filter layer.

Weeps should consist of 6-inch diameter pipes having a maximum horizontal spacing of 6.0 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or filter fabric of a gage that will prevent passage of the gravel filter layer. Alternatively, a closed end pipe with horizontal slits can be used for the drain; in this case, the slits must be of a size that will not pass the granular filter material.

11.7.5.2 Construction

The following construction procedures and specifications are recommended:

- Normal construction procedures include (a) bank clearing and grading; (b) development of a foundation; (c) trenching and setting forms for stubs; (d) placing the filter layer; (e) forming for and placing the concrete pavement (including any special adaptations necessary for the revetment toe); (f) backfilling toe trenches (if required); and (g) vegetation of disturbed areas.
- The usual specifications for placing and curing structural concrete should apply to concrete slope paving.
- Subgrade should be dampened before placement of the concrete
- Reinforcement must be supported so that it will be maintained in its proper position in the completed paving.
- If the slope is too steep to allow ordinary hand finishing, a 3-inch thickness of mortar may be applied immediately after the concrete has set.
- Slabs should be laid in horizontal courses, with cold joints without filler between courses. These joints should be formed with 4-inch lumber, which should be removed and the joint left open upon completion.
- Vertical expansion joints should run normal to the bank at 15- to 20 ft intervals. These joints should be formed using joint filler.
- Headers or forms for use during screening or rodding operations must be firm enough and so spaced that adjustment will not be necessary during placement operations.

11.7 Design Guidelines (continued)

11.7.5.2 Construction (continued)

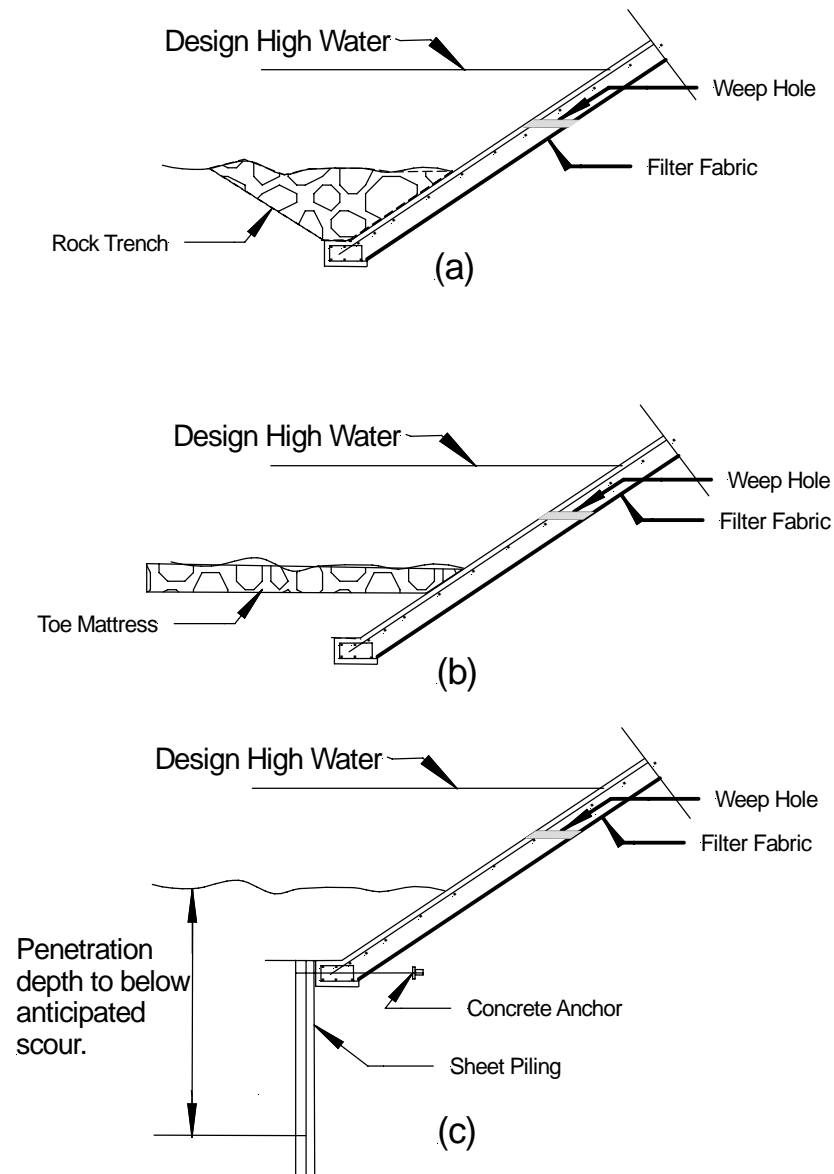


Figure 11-35 Concrete Pavement Toe Details.

11.7 Design Guidelines (continued)

11.7.6 Grouted Fabric Slope Paving

Grouted Fabric-formed revetments are a relatively new development for use on earth surfaces subject to erosion. They have been used as an alternative to traditional revetments such as concrete liners or riprap on reservoirs, canals and dikes.

Grouted fabric-formed revetments are made by pumping a highly fluid structural grout, often referred to as "fine aggregate concrete," into an insitu envelope consisting of a double-layer synthetic fabric. During filling, excess mixing water is squeezed out through the highly permeable fabric substantially reducing the water/cement ratio with consequent improvement in the quality of the hardened concrete. A major advantage of this type revetment is that fabric formed revetments may be as easily assembled underwater as in a dry location.

There are three commonly used types of fabric-formed revetments.

Type 1 Two layers of nylon fabric are woven together at 4-to 6-inch centers as indicated in Figure 11-36. These points of attachment serve as filter points to relieve hydrostatic uplift caused by percolation of ground water through the underlying soil. The finished revetment has a deeply cobbled or quilted appearance. Mat thickness typically average from 2- to 6-inch.

Type 2 Two layers of nylon or polypropylene woven fabric are joined together at spaced centers by means of interwoven tie cords, the length of which controls the thickness of the finished revetment. Plastic tubes may be inserted through the two layers of fabric prior to grout injection to provide weep holes for relief of hydrostatic uplift. The finished revetment is of relatively uniform cross section and has a lightly pebbled appearance. Mat thickness typically averages from 2- to 10 inches.

Type 3 Two layers of nylon fabric are interwoven in a variety of rectangular block patterns, the points of interweaving serving as hinges to permit articulation of the hardened concrete blocks. Revetments are reinforced by steel cables or nylon rope threaded between the two layers of fabric prior to grout injection and remain embedded in the hardened cast-in-place blocks. Block thickness is controlled by spacer cords in the middle of each block.

11.7.6.1 Design Guidelines

The specially woven fabric for grouted fabric formed revetments are manufactured by several different companies. The designer should consult with the manufacturer's literature for designing and selecting the appropriate type of material and thickness for a given hydraulic condition.

11.7 Design Guidelines (continued)

11.7.6.1 Design Guidelines (continued)

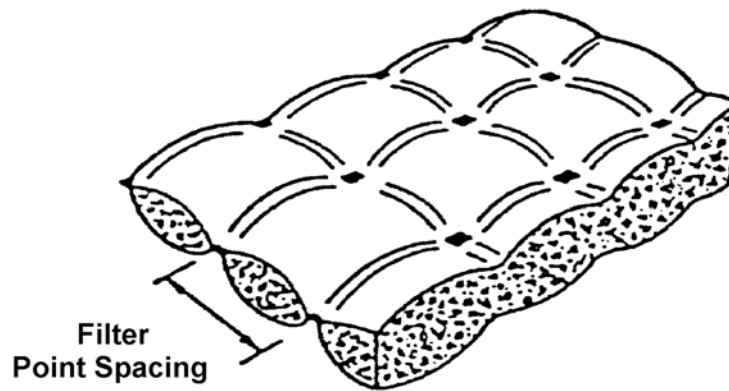


Figure 11-36 Type 1 Grouted Fabric-Formed Revetment

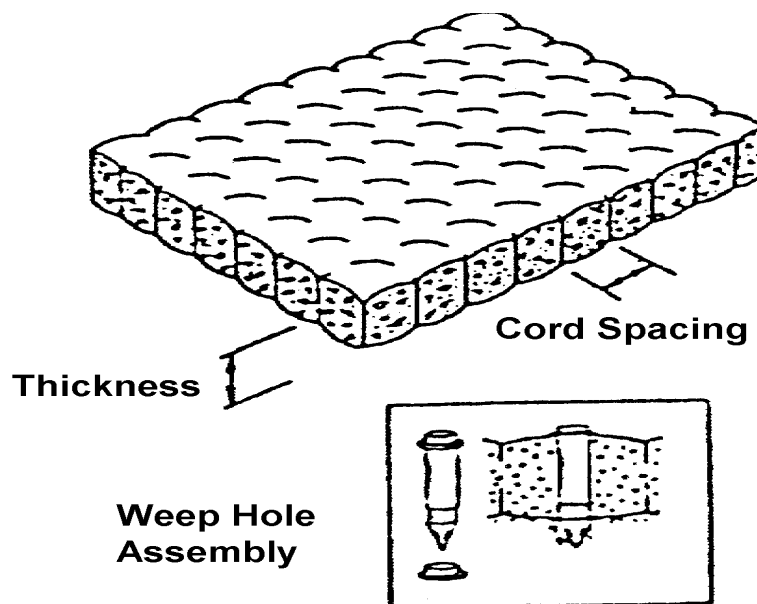


Figure 11-37 Grouted Fabric-Formed Revetment

11.8 References

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Appendix A– Geotextile Design and Selection Criteria

Application Evaluation

Step 1. Determine if design requirements are critical and/or severe

A. Critical/Noncritical

1. If the erosion control system fails will there be a risk of loss of life?
2. Does the erosion control system protect a significant structure and will failure lead to significant structural damage.
3. If the geotextile clogs, will failure occur with no warning? Will failure be catastrophic?
4. If erosion control system fails, will the repair costs greatly exceed installation costs?

B. Severe/Nonsevere

1. Are soils to be protected gap graded or pipable soils?
2. Are soils present which consists of primarily silts and uniform sands with 85% passing the #100 sieve?
3. Will the erosion control system be subjected to reversing or cyclic flow conditions such as wave action or tidal variations?
4. Will high gradients exist in the soil to be protected? Will rapid drawdown conditions or seeps or weeps in the soil exist, and whose blockage would produce high hydraulic pressures?
5. Will high velocity conditions exist such as in stream channels?

NOTE: IF the answer is yes to any of the above questions, the design should proceed under the critical/severe Requirements; otherwise use the noncritical/nonsevere design approach.

Step 2. Identify in-place (backfill) Material Properties

A. Grain size analyses.

1. Obtain D_{85} for each soil and select worst soil for retention (i.e., soil with smallest D_{85}).
2. Calculate $C_u = d_{60} / d_{10}$. Note: When the protected soil contains particles from 1 inch to those passing the #200 sieve, use only the gradation of soil passing the No. 4 sieve in selecting the geotextile (i.e. scalp off the +No. 4 material).

B. Permeability test

1. Select the worse case soil (i.e., soil having the highest coefficient of permeability, k). Permeability of clean sands with $0.1 \text{ mm} < d_{10} < 3 \text{ mm}$ and $C_u < 5$ can be estimated by Hazen's formula, $k = d_{10}^2 (k \text{ in cm/sec; } d_{10} \text{ in mm})$. This formula is not to be used for fine-grained soils.

Appendix A– Geotextile Design and Selection Criteria(continued)

Application Evaluation

Step 3 Evaluate Armor Material

A. Size Armor Stone

When minimum size of stone exceeds 4 inches or greater than a 4 inch gap exists between blocks, an intermediate layer 6 inches thick should be used between the armor stone and geotextile. Gravel should be sized such that it will not wash through the armor stone ($d_{85} \text{ gravel} > d_{15} \text{ riprap}/5$).

B. Determine Armor Stone Placement Technique. Design Requirements:

I. Soil Retention

Soils	Steady State Flow	Dynamic Pulsating, and Cyclic Flow
<50% passing US #200 sieve	AOS or $0_{95} \leq BD_{85}$	
	$C_u \leq 2$ or ≥ 8 ; $B=1$	
	$2 \leq C_u \leq 4$; $B=0.56C_u$	$0_{95} \leq 0.5 D_{85}$
	$4 \leq C_u \leq 8$; $B=8/C_u$	
>50% Passing	Woven: $0_{95} \leq D_{85}$	$0_{95} \leq 0.5 D_{85}$
	Nonwoven: $0_{95} \leq 1.8 D_{85}$	
	$0_{95}(\text{fabric}) \leq 0.3\text{mm (No. 50 sieve)}$	

NOTE: 1. When the protected soil contains particles for 1-inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.

2. Select fabric on the basis of largest opening value required (smallest AOS).

Appendix A– Geotextile Design and Selection Criteria (continued)**Design Requirements:****II. Permeability****A. Critical/Sever Applications:**

$$k(\text{fabric}) \geq 10k(\text{soil})$$

B. Less critical/Less Severe Applications (with clean medium to coarse sands and gravels)

$$k(\text{fabric}) \geq k(\text{soil})$$

NOTE: Permeability should be based on the actual fabric open area available for flow. For example, if 50% of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50%.

III. Clogging Criteria**A. Critical/Severe Applications**

Select fabrics meeting I, II, IIIB and perform soil/fabric filtration test before specification, prequalifying the fabric, or selection before bid opening. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ratio $\leq 3B$

B. Less critical/Nonsevere Applications

1. Perform soil/fabric filtration tests.
2. Alternative: $0.95 > 3d_{15}$ for $C_u > 3$
3. For $C_u \leq 3$, fabric with maximum opening size possible (lowest AOS) from retention criteria should be specified.
4. Apparent Open Area Qualifiers²

Woven Fabrics: Percent Open Area: $\geq 4\%$

Nonwoven Fabrics: Porosity² $\geq 30\%$

NOTE: 1. Filtration test are performance test and cannot be performed by the manufacturer as tests depend on specific soil and design conditions. Test to be performed by specifying agency or representative.

NOTE: Experience required to obtain reproducible results in gradient ratio test.

2. Porosity requirement based on graded filter porosity.

Appendix A– Geotextile Design and Selection Criteria (continued)**Design Requirements:****IV Survivability Requirements**

Physical Requirements For Erosion Control Geotextile
(From AASHTO-AGC-ARTBA Task Force 25)

Property	Class A	Class B	Test Method
Grab Strength, lbs	200	990	ASTM D4632
Elongation, % (min)	15	15	ASTM D4632
Seam Strength, lbs	180	80	ASTM D4632
Puncture Strength, lbs	80	40	ASTM D4833
Burst Strength, psi	320	140	ASTM D3787
Trapezoidal Shear, lbs	50	30	ASTM D3787
Ultraviolet Degradation at 150 hours	70% Strength retained for all Classes		ASTM D4355

NOTE:

1. Acceptance of geotextile material is to be based on TF25 ACCEPTANCE/REJECTION Guidelines (ASTM D4759)
2. ADOT may require a letter from supplier certifying that its geotextile meets specification requirements.
3. Minimum - Use material in weaker principal direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in lot shall meet or exceed the minimum values in the table). Stated values for non-critical, non-sever condition. Lot sampled according to ASTM D4354.
4. Class A erosion Control applications are those where fabrics are used under conditions where installations stresses are more severe than Class B. (i.e., stone placement height should be less than 3 feet and stone weights should not exceed 250 pounds.)
5. Class B Erosion Control applications are those where fabric is used in structures or under conditions where the fabric is protected by a sand cushion or by “zero drop height” placement of stone.
6. Values apply to both field and manufactured seams.